TEST AND INVESTIGATION GUIDE
continued

NDE 2

Limitations

The properties of the wall can influence the usefulness of sounding. The geometry of the wall and the thickness of the wall will affect the results (ASCE, 1990). Sounding is best used away from the perimeter of the wall and on a wall of uniform thickness.

The accuracy of information from sounding with a hammer also depends on the skill of the engineer or technician performing the test and on the depth of damage within the thickness of the wall. Delaminations up to the depth of the cover for the reinforcing bars (usually about 1 to 2 inches) can usually be detected. Detection of deeper spalls or delamination requires the use of other NDE techniques. Sounding cannot determine the depth of the spall or delamination (Poston et al., 1995).

Tapping on a loose section of material can cause the piece to become dislodged and fall. Avoid sounding overhead. A ladder, scaffold, or other lift device should be used to reach higher elevations of a wall.

References

ACI Committee 224, 1994, Causes, Evaluation and Repair of Cracks in Concrete Structures", ACI Committee 224.1R-93, *Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

ASCE, 1990, *Guideline for Structural Condition Assessment of Existing Buildings*, ASCE 11-90, American Society of Civil Engineers, New York, New York.

Poston, R.W, A.R. Whitlock, and K.E. Kesner, 1995, "Condition Assessment Using Nondestructive Evaluation" *Concrete International*, July, 1995, American Concrete Institute, Detroit, Michigan, pp 36-42.

NDE 3	TEST AND INVESTIGATION GUIDE REBOUND HAMMER	Test Type: **Nondestructive** Materials: **Concrete, Unreinforced Masonry**

Description

A rebound hammer provides a method for assessing the in-situ compressive strength of concrete. In this test, a calibrated hammer impact is applied to the surface of the concrete. The amount of rebound of the hammer is measured and correlated with the manufacturer's data to estimate the strength of the concrete. The method has also been used to evaluate the strength of masonry.

Equipment

A calibrated rebound hammer is a single piece of equipment that is hand operated

Execution

ASTM C805 (ASTM, 1995) provides a standard on the use of a rebound hammer. The person operating the equipment places the impact plunger of the hammer against the concrete and then presses the hammer until the hammer releases. The operator then records the value on the scale of the hammer. Typically three or more tests are conducted at a location. If the values from the tests are consistent, record the average value. If the values vary significantly, additional readings should be taken until a consistent pattern of results is obtained.

Since the test is relatively rapid, a number of test locations can be chosen for each wall. The values from the tests are converted into compressive strength using tables prepared by the manufacturer of the rebound hammer.

Personnel Qualifications

A technician with minimal training can operate the rebound hammer. An engineer experienced with trebound hammer data should be available to supervise to verify that any anomalous values can be explained.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall, indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test marked on either a floor plan or wall elevation.
- Record the number of tests conducted at a given location.
- Report either the average of actual readings or the average values converted into compressive strength along with the method used to convert the values into compressive strength.
- Report the type of rebound hammer used along with the date of last calibration.
- Record the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE
continued

NDE 3

Limitations

The rebound hammer does not give a precise value of compressive strength, but rather an estimate of strength that can be used for comparison. Frequent calibration of the unit is required (ACI, 1994). Although manufacturers' tables can be used to estimate the concrete strength, better estimates can be obtained by removing core samples at selected locations where the rebound testing has been performed. The core samples are then subjected to compression tests. The rebound values from other areas can be compared with the rebound balues that correspond to the measured core compressive strength.

The results of the rebound hammer tests are sensitive to the quality of the concrete on the outer several inches of the wall (Krauss, 1994). More reproducible results can be obtained from formed surfaces rather than from finished surfaces.

Surface moisture and roughness can also affect the readings. The impact from the rebound hammer can produce a slight dimple in the surface of the wall. Do not take more than one reading at the same spot, since the first impact can affect the surface, and thus affect the results of a subsequent test.

When using the rebound hammer on masonry, the hammer should be placed at the center of the masonry unit. The values of the tests on masonry reflect the strength of the masonry unit and the mortar (Noland et al., 1982). This method is only useful in assessing the strength of the outer wythe of a multi-wythe wall.

References

ACI Committee 364, 1994, "Evaluation of Structures Prior to Rehabilitation", ACI 364.1R, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan

ASTM, 1995, *Test for Rebound Number of Hardened Concrete*, ASTM C805, American Society for Testing and Materials, Washington, DC

Krauss, P.D., 1994, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, NRC JNC No. B8045, US Nuclear Regulatory Commission, Washington, DC.

Noland, J.L. et al., 1982, *An Investigation into Methods of Nondestructive Evaluation of Masonry Structures*, Report to the National Science Foundation, Atkinson-Noland & Associates, Boulder, Colorado

TEST AND INVESTIGATION GUIDE	Test Type:	**Nondestructive**
NDE 4 — **REBAR DETECTOR**	Materials:	**Concrete, Reinforced Masonry**

Description
Covermeter is the general term for a rebar detector used to determine the location and size of reinforcing steel in a concrete or masonry wall. The basic principle of most rebar detectors is the interaction between the reinforcing bar and a low frequency magnetic field. If used properly, many types of rebar detectors can also identify the amount of cover for the bar and/or the size of the bar. Rebar detection is useful for verifying the construction of the wall, if drawings are available, and in preparing as-built data if no previous construction information is available.

Equipment
Several types and brands of rebar detectors are commercially available. The two general classes are those based on the principle of magnetic reluctance and those based on the principle of eddy (Carino, 1992). The various models can have a variety of features including analog or digital readout, audible signal, one-handed operation, and readings for reinforcing bars and prestressing tendons. Some models can store the data on floppy disks to be imported into computer programs for plotting results.

Execution
The unit is held away from metallic objects and calibrated to zero reading. After calibration, the unit is placed against the surface of the wall. The orientation of the probe should be in the direction of the rebar that is being detected. The probe is slid slowly along the wall, perpendicular to the orientation of the probe, until an audible or visual spike in the readout is encountered. The probe is passed back and forth over the region of the spike to find the location of the maximum reading, which should correspond to the location of the rebar. This location is then marked on the wall. The procedure is repeated for the perpendicular direction of reinforcing.

If size of the bar is known, the covermeter readout can be used to determine the depth of the reinforcing bar. If the depth of the bar is known, the readout can be used to determine the size of the bar. If neither quantity is known, most rebar detectors can be used to determine both the size and the depth using a spacer technique. The process involves recording the peak reading at a bar and then introducing a spacer of known thickness between the probe and the surface of the wall. A second reading is then taken. The two readings are compared to estimate the bar size and depth.

Intrusive testing can be used to help interpret the data from the detector readings. Selective removal of portions of the wall can be performed to expose the reinforcing bars. The rebar detector can be used adjacent to the area of removal to verify the accuracy of the readings.

Personnel Qualifications
The personnel operating the equipment should be trained and experienced with the use of the particular model of covermeter being used and should understand the limitations of the unit.

TEST AND INVESTIGATION GUIDE
Continued

NDE 4

Reporting Requirements
The personnel conducting the tests should provide a sketch of the wall indicating the location of the testing and the findings. The sketch should include the following information:

- Mark the locations of the test on either a floor plan or wall elevation.
- Report the results of the test, including bar size and spacing and whether the size was verified.
- List the type of rebar detector used.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations
The readings can be difficult to interpret if the depth of the reinforcement is too great or if there is heavy congestion of reinforcement, such as at splices or boundaries (ACI, 1994). The accuracy will vary between units and manufacturers. Except at the boundaries of the wall, the spacing of bars is generally wide enough that the influence of adjacent bars should not affect the readings. Other embedded metals, such as metallic conduits or pipes will be detected and may give false readings.

For walls with two layers of reinforcing steel, the rebar detector can only be used to detect the reinforcing bars closest to the face on which the probe is used. The unit should be used on both faces to detect bars in a wall with two layers of reinforcement. When two layers are present, the second layer of reinforcement can affect the readings by producing a stronger signal for a bar of given size and depth than the bar would produce in the absence of a second layer (ACI, 1997).

Some rebar detectors require recalibration at regular intervals during use. Therefore, the user should frequently check the readings to verify that the readings are still reproducible. The spacer technique to determine the size and depth of reinforcement is only accurate to within 1 or 2 bar diameters (Krauss, 1994 and Bungey, 1989).

When measuring the cover depth, many units actually measure the distance to the center of the reinforcing steel. The manufacturer's literature should be reviewed to determine the meaning of the depth reading.

References
ACI Committee 228, 1997, *Nondestructive Tests Methods for Evaluation of Concrete in Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

ACI Committee 364, 1994, Guide for Evaluation of Concrete Structures Prior to Rehabilitation, ACI 364.1R, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

Bungey, J.H., 1989, *Testing Concrete in Structures*, 2nd Edition, Chapman and Hall, New York, New York.

Carino, N.J, 1992, *Performance of Electromagnetic Covermeters for Nondestructive Assessment of Steel Reinforcement*, NISTR 4988, National Institute of Standards and Technology, Gaithersburg, Maryland.

Krauss, P.D., 1994, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, NRC JNC No. B8045, US Nuclear Regulatory Commission, Washington, DC.

NDE 5	TEST AND INVESTIGATION GUIDE	Test Type:	**Nondestructive**
	ULTRASONIC PULSE VELOCITY	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description
The ultrasonic pulse velocity method measures the travel time of an ultrasonic pulse through the thickness of the wall. The velocity at which the pulse travels through the wall is affected by the quality of the material, including the presence of cracking or damage. By comparing the relative travel time at various sections of known thickness, the ultrasonic pulse velocity can be used to assess relative strength of concrete or masonry and to indicate the presence of cracking or delamination.

Equipment
The equipment and calibration procedures are described in ASTM C 597, *Standard Test Method for Pulse Velocity Through Concrete*. Portable equipment is available from several manufacturers. Transmitting and receiving transducers are required.

The frequency of the transducers is typically about 50 kHz, which is adequate for walls that are at least four inches thick, corresponding to the wave length of the pulse (Krauss, 1994).

A time-measuring meter with either a digital time display or a digital storage oscilloscope is also required.

Execution
The area of the wall to be examined should be laid out with a grid. The location of the grid should be coordinated so that the intersection of the grid lines will be at the same location on both sides of the wall. The spacing of the grid will vary, depending on the size of the wall and the extent of the expected damage. A grid spacing of one-foot centers should provide a reasonably fine spacing to capture potential damage.

The transmitting and receiving transducers are mounted on opposite sides of the wall, using a couplant between the transducer and the surface of the wall. For masonry walls, the transducer should be mounted to the masonry units, not to the mortar joints. The meter sends a series of pulses through the wall and measures the transmission time, which is recorded. The transducers are moved to the next location, and the test is repeated. If high readings are encountered, indicating possible discontinuities, a finer grid should be laid out in the vicinity of the possible damage to eastablish the extent of the discontinuity.

The results are displayed as travel time. The travel time needs to be converted into velocity using the thickness of the wall. The pulse velocity, not the travel time, should be used to compare results at various locations.

To establish the relative strength of the wall, samples should be taken at representative test locations. Correlate the strength of the extracted material samples and the pulse velocity readings at those locations, then use this correlation to estimate the strength at other sections of the wall.

Personnel Qualifications
A technician with training in the use of the equipment can carry out the test. An engineer or technician with extensive experience in the use and limitations of the equipment should be responsible for overseeing the tests and interpreting the results.

TEST AND INVESTIGATION GUIDE
Continued

NDE 5

Reporting Requirements

The personnel conducting the tests should provide a sketch of the wall indicating the location of the testing and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Report the test results as either actual velocity measurements or interpreted results.
- List the type of pulse-velocity equipment used, including the date of last calibration.
- Record the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

Pulse-velocity measurements require access to both sides of the wall. The wall surfaces need to be relatively smooth. Rough areas can be ground smooth to improve the acoustic coupling. Couplant must be used to fill the air space between the transducer and the surface of the wall. If air voids exist between the transducer and the surface, the travel time of the pulse will increase, causing incorrect readings.

Some couplant materials can stain the wall surface. Non-staining gels are available, but should be checked in an inconspicuous area to verify that it will not disturb the appearance.

Embedded reinforcing bars, oriented in the direction of travel of the pulse, can affect the results, since the ultrasonic pulses travel through steel at a faster rate than through concrete. Bars larger than 3/8-inch diameter will significantly affect the results (Chung and Law, 1983). The moisture content of the concrete also has a slight effect (up to about 2 percent) on the pulse velocity.

Pulse-velocity measurements can detect the presence of voids or discontinuities within a wall; however, these measurements cannot determine the depth of the voids (ACI, 1997).

References

ACI Committee 228, 1997, *Nondestructive Tests Methods for Evaluation of Concrete in Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

ACI Committee 364, 1994, Guide for Evaluation of Concrete Structures Prior to Rehabilitation, ACI 364.1R, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

Berra, M., L. Binda, G. Baronio, and A. Fatticcioni, 1987, "Ultrasonic Pulse Transmission: A Proposal to Evaluate the Efficiency of Masonry Strengthened by Grouting", *Evaluation and Retrofit of Masonry Structures*, Proceedings of the Second Joint USA-Italy Workshop on Evaluation and Retrofit of Masonry Structures, pp 93-110.

Chung, H.W. and K.S. Law, 1983, "Diagnosing In Situ Concrete by Ultrasonic Pulse Technique", *Concrete International*, October 1983, American Concrete Institute, Detroit, Michigan, pp 42-49.

Krauss, P.D., 1994, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, NRC JNC No. B8045, US Nuclear Regulatory Commission, Washington, DC.

NDE 6	TEST AND INVESTIGATION GUIDE	Test Type:	**Nondestructive**
	IMPACT ECHO	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description

Impact echo is a method for detecting discontinuities within the thickness of a concrete or masonry wall. The surface of the material is struck with an impactor, a small hammer, which introduces an energy pulse into the material. The energy pulse is reflected off of wave-speed discontinuities within the material. The discontinuities can be cracks, the back surface of the material, side surfaces, delaminations, or voids. A transducer mounted to the striking surface records the reflection of the energy. The transducer is connected to a Fast Fourier Transform (FFT) analyzer, which converts the time history signal from the transducer into the frequency domain. The frequency results and the raw time history can be interpreted to assess the thickness of the material and the size and location of discontinuities within the wall, such as voids, cracks, and delaminations.

Equipment

The typical impact echo equipment consists of a small impactor (hammer) to strike the surface of the material, a transducer to measure the surface response of the material, and an FFT analyzer to analyze the measurements made by the transducer. Some FFT analyzers are extremely sophisticated portable computers that allow for extensive manipulation of the output, while others simply provide a graph showing the frequency content of the output. The equipment can be assembled from available components. Complete systems are also commercially available (ACI, 1997).

Execution

The transducer is placed on the surface of the material. Good contact must be developed between the transducer and the material, or the transducer will not be able to get a clean signal from the energy pulse. Strike the material with the impactor to introduce an energy pulse into the material. The FFT analyzer produces a frequency content analysis of the transducer's signal, but the final analysis of the data rests with the operator. If the impactor and transducer are in the middle of a solid wall, the energy pulse bounces back and forth between the front and back surfaces, typically giving an FFT frequency content with one major frequency peak. This peak corresponds to how quickly the energy pulse bounces between the front and back surfaces of the material. If the impactor and transducer are in the middle of a wall with a delamination, the first peak should be at a higher frequency, since the energy pulse will tend to bounce between the front surface of the wall and the surface of the delamination, a shorter distance requiring less travel time. The presence of side boundaries, voids, large concentrated amounts of reinforcement, and cracks will complicate the signal.

Personnel Qualifications

Impact echo testing should be performed by an engineer or technician well-trained and experienced in using this technique.

TEST AND INVESTIGATION GUIDE
continued

NDE 6

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the tests and the findings. The sketch should include the following information:

- Mark location of the test on either a floor plan or wall elevation.
- Record the number of tests conducted at a given location.
- Report the results of the test using either the actual readings or the interpreted results, including the peak frequency values.
- Describe the type of impact echo equipment used, along with the date of last calibration.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

The accuracy of impact echo testing is typically highly dependent on the skill of the engineer or technician in understanding the testing method and interpreting the results. Incompletely trained or untrained persons using impact echo methods have a high probability of interpreting the results incorrectly. The physical limitations and accuracy of impact echo are governed in part by the size of the impactor, the type, sensitivity, and natural frequency of the transducer, the uniformity of the concrete, and the ability of the FFT analyzer to manipulate the data into useful information.

The impact echo technique has been applied extensively to concrete structures. However, there is little experience with applying the technique to reinforced or unreinforced masonry components.

References

ACI Committee 228, 1997, *Nondestructive Test Methods for Evaluations of Concrete in Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

Poston, R.W, et al., 1995, "Condition Assessment Using Nondestructive Evaluation" *Concrete International*, July, 1995, American Concrete Institute, Detroit, Michigan, pp 36-42.

	TEST AND INVESTIGATION GUIDE	Test Type: **Nondestructive**
NDE 7	**SPECTRAL ANALYSIS OF SURFACE WAVES (SASW)**	Materials: **Concrete**

Description

Spectral analysis of surface waves (SASW) is a method of measuring the propagation of surface waves over a wide range of wavelengths. The propagation velocities are measured using accelerometers and a Fast Fourier Transform (FFT) analyzer. The results can be interpreted to assess the thickness of the material and the size and location of discontinuities within the wall, such as voids, large cracks, and delaminations.

Equipment

The SASW tests require the following equipment:

- An impactor, which is usually a hammer
- Two or more receivers, which could be accelerometers or velocity transducers
- An FFT spectrum analyzer for recording and analyzing the input signal from each receiver

Execution

Mount the receivers on the surface of the wall using a removable adhesive. The spacing between the receivers will depend on the thickness of the wall. Strike the surface of the wall with the hammer away from the receivers, producing a surface R-wave that propagates along the surface (ACI, 1997). The surface velocity or acceleration is recorded by the receivers and processed. The processed results can then be interpreted to assess the condition of the concrete.

Personnel Qualifications

Use of the SASW equipment should be limited to those with extensive training in the use of the equipment. Specialized experience is required to interpret the results.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Record the number of tests conducted at a given location.
- Report the results of the test using either the actual readings or the interpreted results.
- Describe the type of equipment used.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE **continued**	NDE 7

Limitations

The signal processing equipment used for the interpretation of the results is very complex and not readily available. The SASW process has been used mainly on pavement, slabs, and other horizontal surfaces. Its use on walls has not been documented extensively.

References

ACI Committee 228, 1997, *Nondestructive Tests Methods for Evaluation of Concrete in Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

NDE 8	TEST AND INVESTIGATION GUIDE	Test Type:	Nondestructive
	RADIOGRAPHY	Materials:	**Concrete, Reinforced Masonry**

Description

Radiography can be used to determine the location of reinforcing steel within a concrete or masonry wall. The process involves transmitting x-rays through the concrete. A radiographic film on the opposite side from the x-ray source records the intensity of the x-rays that exit the wall. The processed film presents an image of the locations of reinforcing bars and other discontinuities.

Equipment

A portable X-ray tube with a radioactive isotope is required. For wall thickness less than six inches, iridium-192 or cesium-137 can be used (see, for example, Mitchell et al., 1979). For thicker materials, more intense isotopes are needed. Special photographic film is used to capture the X-rays.

Execution

An x-ray technician mounts the photographic film on the surface of the wall. The location of the film is marked on the wall for future reference. The X-ray tube is mounted or placed on the opposite side of the wall from the film. The x-ray technician exposes the wall to the radioactive isotope. The length of time for the exposure will depend on the thickness of the wall, the size of the film, and the amount of reinforcement in the wall. Thicker walls and areas with a high concentration of reinforcing bars require longer exposure times.

The film is then processed. The processed image is interpreted to assess the locations of reinforcing bars and discontinuities. Reinforcing bars, which are denser than concrete, show up on the image as light areas. Voids are seen on the film as relatively darker areas (ACI, 1997).

Personnel Qualifications

The personnel operating the x-ray equipment require highly specialized training on the handling of radioactive material. The technicians who interpret the images should be experienced in viewing x-rays from concrete or masonry structures.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Report the results of the test along with a sketch of the findings.
- Describe the type of X-ray equipment used, along with the date of last calibration.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE
continued

NDE 8

Limitations

Because radiography involves the release of radiation, the vicinity of the testing needs to be evacuated, except for the personnel conducting the tests. The size of the area to be evacuated depends on the type of radioactive isotope used and the thickness of the wall. Thicker walls require longer exposure times, and therefore more radiation is released. The time, expense, and logistics of the evacuation must be considered in planning the tests. Most commercially available x-ray equipment is capable of penetrating walls up to 12 inches thick. For thicker walls, the expense of the highly specialized equipment needed is generally not cost-effective for commercial buildings.

The presence of steel within the concrete will produce a shadow on the film to indicate its location. Since the radiation emits from a point source onto the photographic film, the amount of shadow will depend on the depth of the reinforcing bar from the face where the x-ray source is placed. Therefore, it is usually not possible to determine the size of the reinforcing bars based on the photographic image.

References

ACI Committee 228, 1997, *Nondestructive Test Methods for Evaluation of Concrete Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

Mitchell, T.M., P.L. Lee, and G.J. Eggert, 1979, "The CMD: A Device for the Continuous Monitoring of the Consolidation of Plastic Concrete, *Public Roads*, Vol. 42, No. 148.

TEST AND INVESTIGATION GUIDE	Test Type:	**Nondestructive**
NDE 9 — PENETRATING RADAR	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description

Penetrating radar transmits electromagnetic waves, which are received by an antenna. The propagation of the waves through the material is influenced by the dielectric constant and the conductivity of the material. The signal received can be interpreted to discover discontinuities and variations in the material properties. The interpreted data can be used to detect the location of reinforcing bars, cracks, voids, or other material discontinuities.

Equipment

The penetrating radar instrumentation consists of several components including:

- An antenna that emits an electromagnetic pulse of various frequencies
- A receiving antenna
- A control unit that provides power to the transmitting antenna and acquires the signal from the receiving antenna
- A data recording device such as a printed display or digital storage device

Execution

Place the antenna of the radar unit on the surface of the wall and move along the surface while data are being recorded. The antenna produces an electromagnetic pulse that passes through the wall. Some of the pulse is reflected back to the receiving antenna. The received signal is printed on a strip-recording chart or stored for later analysis (Mellett, 1992). The recorded data represent the condition along the length of the wall for the width of the antenna. Multiple passes are required to obtain data for widths greater than the width of the antenna.

The data can then be interpreted to evaluate the location and depth of reinforcing steel, the thickness of the wall, and the location of delaminations or voids.

Personnel Qualifications

Use of the penetrating radar equipment should be limited to those with the extensive training required to correctly interpret the results (ACI, 1994).

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Report the results of the test using either the actual recorded data with the interpretations marked on the printed data or the interpreted results only.
- List the type of radar equipment used, including the type of antenna.
- Report the date of the test.
- List the engineer responsible for interpreting the test results and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE
Continued

NDE 9

Limitations

Although penetrating radar units are commercially available, very few units are in service for use with concrete and masonry structures. For example, less than five units are in use in California. Penetrating radar has been used primarily on slabs-on-grade for detecting subsurface conditions. Some work has been done to apply the method to concrete columns (Delgado and Heald, 1996) and to unreinforced masonry buildings.

A high-frequency antenna provides high resolution, but has shallow penetration, whereas deeper penetration with reduced resolution can be achieved with lower-frequency antennae (Krauss, 1994). Radar cannot effectively detect small differences in materials because the effective resolution is typically one-half of the wavelength (Candor, 1984).

Although penetrating radar is useful for locating the spacing and depth of reinforcing bars, it is not possible to determine the size of the bars. Closely-spaced bars can make it difficult to discern bar locations and depths. Close spacing make it difficult to detect features below the layer of reinforcing steel (ACI, 1997). Large metallic objects, such as embedded steel members cannot be clearly identified because of the scattering of the electromagnetic pulse.

References

ACI Committee 228, 1997, *Nondestructive Tests Methods for Evaluation of Concrete in Structures*, ACI 228.2R - Draft, American Concrete Institute, Detroit, Michigan.

ACI Committee 364, 1994, Guide for Evaluation of Concrete Structures Prior to Rehabilitation, ACI 364.1R, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

Candor, T. R., 1984, "Review of Penetrating Radar as Applied to Nondestructive Evaluation of Concrete", *In Situ/Nondestructive Testing of Concrete*, ACI SP-82, American Concrete Institute, Detroit, Michigan, pp 581-601.

Delgado, M., and S.R. Heald, 1996, "Post-Earthquake Damage Assessed Nondestructively", *Materials Evaluation*, pp 378-382.

Krauss, P.D., 1994, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, NRC JNC No. B8045, US Nuclear Regulatory Commission, Washington, DC.

Mellet, J, 1992, "Seeing Through Solid Materials", *Building Renovation*, Penton Publishing.

TEST AND INVESTIGATION GUIDE	Test Type:	**Intrusive**
IT 1 — SELECTIVE REMOVAL	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description

When information regarding the construction of portions of the concrete or masonry cannot be obtained using nondestructive techniques, selective removal of portions of the wall is sometimes required to allow direct observation of the condition of the reinforcing bars or interior portion of the concrete or masonry. Removal is suggested only when visual observations of the surface indicate that the wall may have hidden damage such as buckled rebar, or when it is necessary to determine the construction of the wall.

Equipment

Light chipping tools, small diameter core drills, or masonry saws are used for creating openings in the wall.

A fiber-optic borescope can be used to view interior spaces through small openings.

Execution

Portions of the wall are removed by chipping, drilling, or sawing to a specified depth of the wall. The inner construction and condition of the wall are then observed visually. A small mirror and flashlight can be used to better view spaces that are difficult to examine, eliminating the need to remove extensive portions of the wall.

When small holes are used, a fiber-optic borescope can be used to view the interior construction and to look for evidence of damage or deterioration (ACI, 1994). Some borescopes can be fitted with a camera to produce photographic documentation of the observations. Some models also have flexible shafts and pivoting viewing heads to allow for multidirectional viewing.

Following the observations, the intrusive openings should be patched with appropriate material.

Personnel Qualifications

Engineers performing selective removal should be experienced with the equipment being used. The engineer should also be familiar with the drawings or other available documentation on the building to understand the expected results of the intrusive observation.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the intrusive openings and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Describe the size and type of opening.
- Specify the maximum strength of the core obtained during the test, in terms of force and in pressure.
- Describe the results of the test by a written description and a sketch or photograph.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE
continued

IT 1

Limitations

If the findings of the intrusive opening are substantially different from what was expected, the engineer should review all of the available information before proceeding.

The use of selected intrusive openings is often performed in conjunction with nondestructive testing procedures. For example, a rebar detector can be used to establish the location of reinforcing bars nonintrusively. Selected locations of reinforcement can then be chipped out to determine the size of the bar and/or depth of the cover. These data are then used to calibrate the rebar detector.

The information gained from observations at selected locations is only applicable to the surveyed areas. Construction with similar appearance and condition may be different. The amount of variability will depend on several factors, including the era of construction, the amount of gravity and lateral load on the wall, and normal variations in construction quality. The locations of the intrusive openings should be carefully chosen to include sufficient typical and atypical areas so that the engineer has confidence that the data gathered represent most of the walls in the building.

Conversely, it is seldom cost effective or necessary to intrusively test every wall in a building. Intrusive openings may damage reinforcing bars or other hidden structural elements. Large intrusive openings can weaken the wall.

References

ACI Committee 364, 1994, "Evaluation of Structures Prior to Rehabilitation", ACI 364.1R, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

TEST AND INVESTIGATION GUIDE	Test Type:	**Intrusive**
IT 2 — PETROGRAPHY	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description

A petrographic evaluation is a microscopic evaluation of the concrete or masonry material. A sample of the material is removed and sent to a laboratory where the sample is prepared and studied using a high-powered microscope. Petrographic examination can also be used to determine the cause of cracking and the approximate mix design of the concrete or mortar.

Equipment

- Typical equipment includes:
- Core drill or other tools for removing concrete or masonry
- Laboratory equipment including concrete saws for sectioning, grinding wheels for polishing, and stereo microscopes

Execution

Remove samples of the concrete or masonry material from the building using core drilling equipment or other concrete removal tools. The samples are then sent to a laboratory where they are cut, polished and examined under a microscope in accordance with ASTM C 856 (ASTM, 1991) procedures.

The condition of the concrete or masonry located along the edge or within a crack can often be used to determine if the crack formed recently and thus may be earthquake related. Some of the methods used to assess the age of cracks are:

- Weathering along cracks. Since the edges of the material on either side of the crack tend to become rounded over time due to normal weathering, it may be possible to estimate whether a crack is "relatively young" or "relatively old" by estimating the amount of weathering.
- Secondary deposits. Secondary deposits within a crack such as mortar, paint, epoxy, or spackling compound indicate that the crack formed before the installation of the material contained within it.
- Interpretation of carbonation patterns. Carbonation of calcium hydroxide contained in hydrated cement paste is inevitable and typically begins along formed or cracked surfaces. Carbonation penetrates into the cementitous material in a direction perpendicular to the plane of the formed or cracked surface. If an estimate of the carbonation rate can be made, then studies of the pattern of cementitious matrix carbonation adjacent to a crack can be used to estimate the age of the crack.

Petrographic studies can also establish the approximate composition of concrete and mortar. This information can be used to establish an approximate material strength. The material composition is needed to formulate or specify compatible materials that will be used for repairs and modifications.

Personnel Qualifications

Much of the results of a petrographic examination are subject to the personal judgment of the petrographer (Krauss, 1994). Therefore, the petrographer must have extensive experience in the evaluation of the materials being tested.

TEST AND INVESTIGATION GUIDE
continued

IT 2

Reporting Requirements

A variety of information is available through petrography. The personnel conducting the tests on the material samples should provide a written report of the findings to the evaluating engineer. The results should contain, at a minimum, the following information for each sample:

- Identify the sample using the description of location or sample number provided by the engineer.
- Specify the length and diameter of the core and the cross-sectional area.
- Describe the tests performed on the sample, along with the appropriate references.
- Describe the results of the examination.
- Report the date the sample was taken and the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

Although petrographic analysis can reveal considerable information regarding the composition of materials and the cause of damage, the results are subjective.

The exact cause and age of cracks may not be discernable. Cracks can have several causes (ACI, 1994). Walls that have been protected with finishes are not subject to the typical surface deterioration that would allow comparison of crack faces for assessing relative age.

References

ACI Committee 201, 1994, "Guide for Making a Condition Survey of Concrete in Service", ACI 201.1R-92, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, Michigan.

ASTM, 1991, *Standard Practice for Petrographic Examination of Hardened Concrete*, ASTM C 856-83, *1991 ASTM Annual Book of Standards*, American Society for Testing and Materials, Philadelphia, PA, pp 416-428.

Krauss, P.D., 1994, *Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants*, NRC JNC No. B8045, US Nuclear Regulatory Commission, Washington, DC.

IT 3	TEST AND INVESTIGATION GUIDE	Test Type:	**Intrusive**
	MATERIAL EXTRACTION AND TESTING	Materials:	**Concrete, Reinforced Masonry, Unreinforced Masonry**

Description

Material testing requires removal of a sample of the material, which can be either the reinforcing steel, concrete, or concrete masonry. The removed samples are then tested to determine the tensile or compressive strength for the steel or concrete, respectively.

Equipment

Concrete cores should be taken with diamond-studded core bits.

Reinforcing steel should be extracted with a reciprocating saw or torch.

Execution

Concrete testing requires removal of core samples with a diamond-tipped drill bit. Typical cores are three to six inches in diameter. For compression testing of the concrete, the length of the cores should be at least two times the diameter (ACI, 1994a). The cores should be taken through sections of walls that have no significant cracking. The core should be taken through the thickness of the wall and should avoid reinforcing steel. The cores should then be prepared and tested in accordance with ASTM C42 procedures (ASTM, 1991a).

Rebar testing requires removal of concrete surrounding a length of reinforcing bar. The length of the sample required is dependent of the size of the bar. The rebar is removed. The removed sample is then subjected to tensile testing. It can also be subjected to metallurgical examination to assess the weldability of the steel. The reinforcing steel sample should be prepared and tested in accordance with ASTM A 370 (ASTM, 1991b).

Following removal of either concrete or rebar samples, the openings should be patched.

Personnel Qualifications

Material samples should be obtained by an experienced contractor. Most areas have contractors that specialize in concrete coring and sawing. The contractor should be familiar with the use of the equipment. An engineer should be responsible for specifying the locations of the sampling. Testing of the samples should be accomplished by a qualified laboratory under the direction of a licensed engineer.

Reporting Requirements

The personnel conducting the tests on the material samples should provide a written report of the findings to the evaluating engineer. The results for the concrete core tests should contain, at a minimum, the following information for each sample:

- Identify the sample using the description of location or sample number provided by the engineer.
- Specify the length and diameter of the core, and cross-sectional area.
- Report the maximum strength of the core obtained during the test, in terms of force and stress.
- Specify the correction factor applied to the results due to the ratio of the length to the diameter, and report the corrected results.
- Report the date the sample was taken and the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

TEST AND INVESTIGATION GUIDE
continued

IT 3

Testing of reinforcing steel should include the following information:

- Identify the sample using the description of location or sample number provided by the engineer.
- Report the length and diameter (or size) of the bar.
- Report the yield and ultimate strength of the core or reinforcing bar obtained during the test, in terms of force and stress.
- Plot the force-elongation data in stress-versus-strain units.
- Report the date the sample was taken and the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

Extraction of samples causes damage to the wall, and repairs to those areas may be required (ACI, 1994b). Therefore, samples should be removed from areas of low expected demands.

The values obtained from the concrete core tests should not be expected to exactly equal the anticipated design strength values. Core test values can have an average value of 85 percent of the specified strength (ACI, 1995). The core strength values should be adjusted using the procedure described in FEMA 274 (ATC, 1997b) to obtain the in-place strength of the concrete. If the results indicate high variability in values, additional samples should be taken and tested to reduce the coefficient of variation. However, the cost of the additional tests should be considered against the benefit of the increased precision of the results.

References

American Concrete Institute, 1994a, Strength Evaluation of Existing Concrete Buildings, ACI 437R-91, *ACI Manual of Concrete Practice*, Part 3, Detroit, Michigan.

American Concrete Institute, 1994b, Evaluation of Structures Prior to Rehabilitation, ACI 364.1R, *ACI Manual of Concrete Practice*, Part 3, Detroit, Michigan.

American Concrete Institute, 1995, *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*, Detroit, Michigan, pg 52.

American Society for Testing and Materials, 1991a, *Standard Test Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*, ASTM C 42-90, 1991 ASTM Standards in Building Codes, Philadelphia, Pennsylvania, pp 27-30.

American Society for Testing and Materials, 1991b, *Standard Test Methods for Mechanical Testing of Steel Products*, ASTM A 370-92, 1991 ASTM Standards in Building Codes, Volume 1, Philadelphia, Pennsylvania, pp 484-529.

ATC, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

ATC, 1997b, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 274, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

IT 4	TEST AND INVESTIGATION GUIDE	Test Type: **Intrusive**
	IN SITU TESTING - IN PLACE SHEAR	Materials: **Unreinforced Masonry**

Description

The shear strength of unreinforced masonry construction depends largely on the strength of the mortar used in the wall. An in-place shear test is the preferred method for determining the strength of existing mortar. The results of these tests are used to determine the shear strength of the wall.

Equipment

- Chisels and grinders are needed to remove the bricks and mortar adjacent to the test area.
- A hydraulic ram, calibrated and capable of displaying the applied load.
- A dial gauge, calibrated to 0.001 inch.

Execution

Prepare the test location by removing the brick, including the mortar, on one side of the brick to be tested. The head joint on the opposite side of the brick to be tested is also removed. Care must be exercised so that the mortar joint above or below the brick to be tested is not damaged.

The hydraulic ram is inserted in the space where the brick was removed. A steel loading block is placed between the ram and the brick to be tested so that the ram will distribute its load over the end face of the brick. The dial gauge can also be inserted in the space.

The brick is then loaded with the ram until the first indication of cracking or movement of the brick. The ram force and associated deflection on the dial gage are recorded to develop a force-deflection plot on which the first cracking or movement should be indicated. A dial gauge can be used to calculate a rough estimate of shear stiffness (Eilbeck et al., 1996).

Inspect the collar joint and estimate the percentage of the collar joint that was effective in resisting the force from the ram. The brick that was removed should then be replaced and the joints repointed.

Personnel Qualifications

The technician conducting this test should have previous experience with the technique and should be familiar with the operation of the equipment. Having a second technician at the site is useful for recording the data and watching for the first indication of cracking or movement. The structural engineer or designee should choose test locations that provide a representative sampling of conditions.

TEST AND INVESTIGATION GUIDE
continued

IT 4

Reporting Results

The personnel conducting the tests should provide a written report of the findings to the evaluating engineer. The results for the in-place shear tests should contain, at a minimum, the following information for each test location:

- Describe test location or give the identification number provided by the engineer.
- Specify the length and width of the brick that was tested, and its cross-sectional area.
- Give the maximum mortar strength value measured during the test, in terms of force and stress.
- Estimate the effective area of the bond between the brick and the grout at the collar joint.
- Record the deflection of the brick at the point of peak applied force.
- Record the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

This test procedure is only capable of measuring the shear strength of the mortar in the outer wythe of a multi-wythe wall. The engineer should verify that the exterior wythe being tested is a part of the structural wall, by checking for the presence of header courses. This test should not be conducted on veneer wythes.

Test values from exterior wythes may produce lower values when compared with tests conducted on inner wythes. The difference can be due to weathering of the mortar on the exterior wythes. The exterior brick may also have a reduced depth of mortar for aesthetic purposes.

The test results can only be qualitatively adjusted to account for the presence of mortar in the collar joints. If mortar is present in the collar joint, the engineer or technician conducting the test is not able to discern how much of that mortar actually resisted the force from the ram.

The personnel conducting the tests must carefully watch the brick during the test to accurately determine the ram force at which first cracking or movement occurs. First cracking or movement indicates the maximum force, and thus the maximum shear strength. If this peak is missed, the values obtained will be based only on the sliding friction contribution of the mortar, which will be less than the bond strength contribution.

References

Eilbeck, D.E, J.D. Lesak, and J.N. Chiropolos, 1996, "Seismic Considerations for Repair of Terra Cotta Cladding", *Proceedings, Seventh North American Masonry Conference,* June, pp 847-858.

ICBO, 1994, In-Place Masonry Shear Tests, UBC Standard 21-6, International Conference of Building Officials, Whittier, California, pg 3-614.

TEST AND INVESTIGATION GUIDE	Test Type: **Intrusive**
IT 5 — IN SITU TESTING - FLAT JACK	Materials: **Unreinforced Masonry**

Description

Flat jacks are thin hydraulic jacks that are inserted into the mortar joints of masonry walls. Flat jacks can be used to measure the state of stress of a masonry wall, the modulus of elasticity of the masonry, and the compressive strength of the masonry.

Equipment

- One or two flat jacks, 1/4- to 3/8-inch thick, with a hydraulic pump and pressure gauge
- Measuring points that are secured to the masonry wall
- Dial calipers or other instruments for measuring the distance between points to within 0.001 inch
- Chipping tools or masonry saws to remove mortar

Execution

A single flat jack is used to determine the state of stress in the masonry. A set of measuring points are attached with epoxy above and below the section of masonry to be tested. The distance between the points is measured and then one horizontal mortar joint is removed with chipping tools or saws. A flat jack is inserted in the mortar joint and pressurized to fill the void. The pressure is increased incrementally while measuring the distance between the measuring points. When the distance between the points returns to the original value, the pressure is recorded and then converted into compressive stress (σ) using the following equation (Rossi, 1987):

$$\sigma = p\, K_m\, K_a$$

Where:

- p is the gauge pressure
- K_m is a jack constant determined by laboratory calibration
- K_a is the ratio of the surface area of the jack to the surface area of mortar removed

A "double flat jack test" is used to evaluate the compressive modulus of elasticity and the compressive strength. A section of the masonry construction to be tested is isolated by cutting two parallel sections of mortar joint. The joints should be separated by approximately the length of the flat jack, typically about 14 inches. Prior to cutting, install measuring points within the section of masonry to be tested. Once the test section has been prepared, insert the flat jacks into the mortar joints and pressurize them to fill the voids. Apply loads to both jacks equally in increments, and measure the distance between the points. If the compressive strength is to be determined, the pressure should be increased until cracking is observed (Kingsley and Noland, 1987a). The pressure and deflection values are then converted into a stress-strain plot for the masonry.

Following the tests, the mortar should be replaced.

Personnel Qualifications

The engineers or technicians conducting the tests should be thoroughly familiar with the use of the equipment and should have experience conducting similar tests.

TEST AND INVESTIGATION GUIDE
continued

IT 5

Reporting Requirements

The personnel conducting the tests should provide a written report of the findings to the evaluating engineer. The report for the flat jack tests should contain, at a minimum, the following information for each test location:

- Describe the test location or use the sample number provided by the engineer.
- For a single flat jack test, report the stress state. For a double flat jack test, report the maximum value for masonry strength, that was measured in the test, in terms of force and pressure.
- Provide the load-deflection curve obtained during the test.
- Report the value of K_m determined by calibration tests.
- Report the date of the test.
- List the responsible licensed engineer overseeing the test and the name of the company conducting the test.

Flat jack tests can be expensive and are prone to several problems.

Limitations

Flat jack tests should be performed in areas that are undamaged. Care should be exercised when removing the mortar for the tests to avoid damaging the mortar or masonry in the test area.

The measuring points must be securely fastened to the masonry units to avoid being dislodged during the testing. The gauge length between the measuring points should be as large as possible so that small changes in movement are easier to detect. The measuring points should be located at the midpoint of the joint and the jack so that the maximum deflections are measured.

If the wall is composed of masonry wythes of differing stiffness (for example, a terra-cotta veneer with brick backup), applying a uniform compressive load using the flat jack may cause out-of-plane bending of the wall. If a flat jack test of the entire wall thickness is performed, measuring points can be attached to both faces of the wall.

Often a flat jack test is performed only on the outer wythe. However, header bricks and mortar in the collar joints may prevent accurate results by restraining the outer wythe. If the masonry units have uneven surfaces, the flat jack will deform into the voids. This causes the pressure to be nonuniformly distributed. An uneven surface also makes it difficult to remove the flat jack from the mortar joint. Also, flat jacks may not have the capacity to fail the masonry in all cases.

References

Kingsley, G.R. and J.L. Noland, 1987a, A Note on Obtaining In-Situ Load-Deformation Properties of Unreinforced Brick Masonry in the United States Using Flatjacks, *Evaluation and Retrofit of Masonry Structures*, Proceedings of the Second Joint USA-Italy Workshop on Evaluation and Retrofit of Masonry Structures, pp 215-223.

Kingsley, G.R. and J.L. Noland 1987b, An Overview of Nondestructive Techniques For Structural Properties of Brick Masonry, *Evaluation and Retrofit of Masonry Structures*, Proceedings of the Second Joint USA-Italy Workshop on Evaluation and Retrofit of Masonry Structures, pp 225-237.

Rossi, P.P., 1987, Recent Developments of the Flat-Jack Test on Masonry Structures, *Evaluation and Retrofit of Masonry Structures*, Proceedings of the Second Joint USA-Italy Workshop on Evaluation and Retrofit of Masonry Structures, pp 257-285.

4. Evaluation of Earthquake Damage

4.1 Basis of Evaluation

The quantitative evaluation of the effects of earthquake damage on structures requires the selection of a measurement parameter. Procedures in this document use change in the anticipated performance of the building during future earthquakes as the measurement parameter. This is the change due directly to effects of earthquake damage on the basic structural properties that control seismic performance. If the structural property changes are estimated, the corresponding change in future performance can also be estimated. The total cost to restore the anticipated performance to approximately that of the building before the damaging earthquake quantifies the effects of the observed damage. These hypothetical "repairs" to structural components are referred to as performance restoration measures.

4.2 Seismic Performance Objectives

The damage evaluation procedures in this document are performance-based; that is, they assess the acceptability of the structural system (and the significance of changes in the structural system) on the basis of the degree to which the structure achieves one or more performance levels for the hazard posed by one or more hypothetical future earthquakes. A performance level typically is defined by a particular damage state for a building. The performance levels defined in FEMA 273, in order of decreasing amounts of damage, are collapse prevention, life safety, and immediate occupancy. Hazards associated with future hypothetical earthquakes are usually defined in terms of ground shaking intensity with a certain likelihood of being exceeded over a defined time period or in terms of a characteristic earthquake likely to occur on a given fault. The combination of a performance level and a hazard defines a performance objective. For example, a common performance objective for a building is that it maintain life safety when subjected to ground motion with a ten-percent chance of exceedance in fifty years.

The damage evaluation begins with the selection of an appropriate performance objective. The performance objective serves as a benchmark for measuring the difference between the anticipated performance of the building in its damaged and pre-event states, that is, relative performance analysis. The absolute performance acceptability of the damaged or pre-event building does not affect the quantification of loss. The quantification of performance loss is affected by the choice of performance objective, as illustrated in the following paragraph. Consequently, the selection of objectives is a matter of policy that depends on the occupancy and use of the facility. Guidance may be found in ATC-40, FEMA 273/274, and FEMA 308.

It is important to note that the damage evaluation procedure can be used to investigate changes in performance characteristics for either single or multiple performance objectives. For example, a hospital might be expected to remain functional (immediate occupancy) after a rare event. For a very rare event, the life safety performance level might be acceptable. The damage evaluation procedure may be used with either or both performance objectives, and the loss associated with the damage may be different for the two objectives. The example hospital might have suffered a $1,000,000 loss, based on the cost of restoration measures, with respect to its ability to remain functional after a rare event. The same level of damage might not have resulted in any loss in its ability to preserve life safety in the very rare event. In summary, the effects of damage can depend greatly on the chosen performance objective.

4.3 Seismic Performance Parameters

Recent research and development activities have resulted in the introduction of structural analysis methodologies based on the inelastic behavior of structures (FEMA 273/274, ATC-40). These techniques generate a plot, called a capacity curve, that relates a global displacement parameter (at the roof level, for example) to the lateral force imposed on the structure. The magnitude of the maximum global displacement to occur during an earthquake depends on elastic and inelastic deformations of the individual components of the structure and their combination into the system response.

For a given global displacement of a structure subject to a given lateral load pattern, there is an associated deformation of each structural component of the building. Since inelastic deformation indicates component damage, the maximum global displacement to occur during an earthquake defines a structural damage state for the building in terms of inelastic deformations for each of its components. The capacity of the structure is represented by the maximum global displacement, d_c, at which the component damage is on the verge of exceeding the tolerable limit for a specific performance level. For example, the collapse prevention capacity of a building might be the roof displacement just short of that at which the associated damage would result in collapse of one or more of the column components. Displacement limits for components are tabulated in FEMA 273 and ATC-40.

The analysis methodologies also include techniques to estimate the maximum global displacement demand, d_d, for a specific earthquake ground motion. The ratio of the displacement capacity, d_c, of the building for a specific performance level to the displacement demand, d_d, for a specific hazard is a measure of the degree to which the building meets the performance objective. If the ratio is less than 1.0 the performance objective is not met. If it is equal to one the objective is just met. If it is greater than 1.0, performance exceeds the objective.

4.4 Relative Performance Analysis

The results of the damage investigation include two related categories of information on the structural damage consequences of the earthquake on the building. First, they comprise a compilation of the physical effects on all of the structural components. These typically consist of cracks in concrete or masonry, spalling or crushing of concrete or masonry, and fracture or buckling of reinforcement. Second, the damage is classified according to component type, behavior mode, and severity. Using these data it is possible for the engineer to quantify the changes attributable to the damage with respect to basic structural properties of the components of the building. These properties include stiffness, strength, and deformation limits.

Damage caused by an earthquake can affect the ability of a structure to meet performance objectives for future earthquakes in two fundamental ways. First, the damage may cause the displacement demand for the future, d'_d, event to differ from that for the pre-event structure, d_d. This is due to changes in the global stiffness, strength, and damping of the structure, which in turn affect the maximum dynamic response of the structure by changing its global stiffness, strength, and damping. Also, the displacement capacity of the damaged structure, d'_c, may differ from that of the pre-event structure, d_c. Damage to the structural components can change the magnitude of acceptable deformation for a component in future earthquakes.

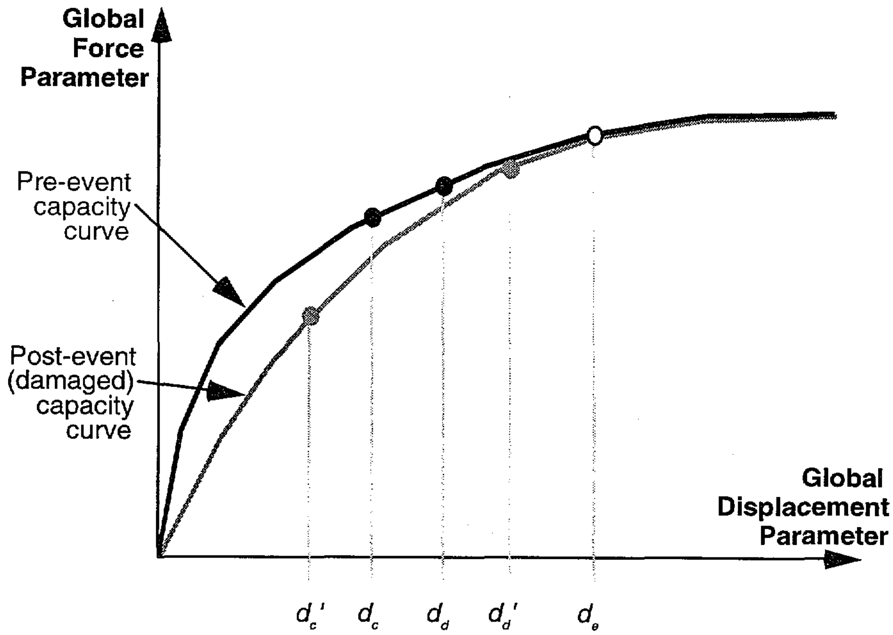
The analysis procedure described in the following sections uses the change in the ability of the damaged building to meet performance objectives in future earthquakes to measure the effects of the damage. The same basic analysis procedure is also used to formulate performance restoration measures that quantify the loss of seismic performance.

4.4.1 Overview

This section summarizes the basic steps of a seismic relative performance analysis for concrete and masonry wall buildings. This is a quantitative procedure that uses nonlinear static techniques to estimate the performance of the building in future events in both its pre-event and damaged states. The procedure is also used to investigate the effectiveness of potential performance restoration measures. This procedure requires the selection of one or more performance objectives for the building as discussed in Section 4.2. The analysis compares the degree to which the pre-event and damaged buildings meet the specified objective. Figure 4-1 illustrates a generalized relationship between lateral seismic forces (base shear or spectral acceleration) and global structural displacements (roof or spectral displacement).

This plot of structural capacity is characteristic of nonlinear static procedures (FEMA 273/274, ATC-40). A point on the curve defines a specific damage state for the structure, since the deformation of all of its components can be related to the global displacement of the structure. Figure 4-2 illustrates the basic idealization of force-deformation characteristics for individual components.

The nonlinear static procedures estimate the maximum global displacement of a stucture to shaking at its base. These procedures are easier to implement and interpret than nonlinear dynamic time history analyses, but they are relatively new and subject to further development. In their present form, they have limitations (Krawinkler, 1996), particularly for buildings that tend to respond in their higher modes of vibration. This limitation,



d_e = Estimate of maximum global displacement caused by damaging earthquake

d_c = Global displacement capacity for pre-event structure for specified performance level

d_c' = Global displacement capacity for damaged structure for specified performance level

d_d = Global displacement demand for pre-event structure for specified seismic hazard

d_d' = Global displacement demand for damaged structure for specified seismic hazard

Figure 4-1 ***Displacement Parameters for Damage Evaluation***

however, is relatively less restrictive for concrete and masonry wall buildings because of their tendency to repond in the fundamental mode. Future development of the procedures may also allow improved treatment for higher modes (Paret et al., 1996). Nonlinear static procedures must be carefully applied to buildings with flexible diaphragms.

The basic steps for using the procedure to measure the effect of damage caused by the damaging ground motion on future performance during the performance ground motion is outlined as follows:

1. Using the properties (strength, stiffness, energy dissipation) of all of the lateral-force-resisting components and elements of the pre-event structure, formulate a capacity curve relating global lateral force to global displacement.
2. Determine the global displacement limit, d_c, at which the pre-event structure would just reach the performance level specified for the performance objective under consideration.
3. For the specified performance ground motion, determine the hypothetical maximum displacement for the pre-event structure, d_d .The ratio of d_c to d_d indicates the degree to which the pre-event structure satisfies the specified performance objective.
4. Using the results of the investigation of the effects of the damaging ground motion, modify the component force-deformation relationships using the

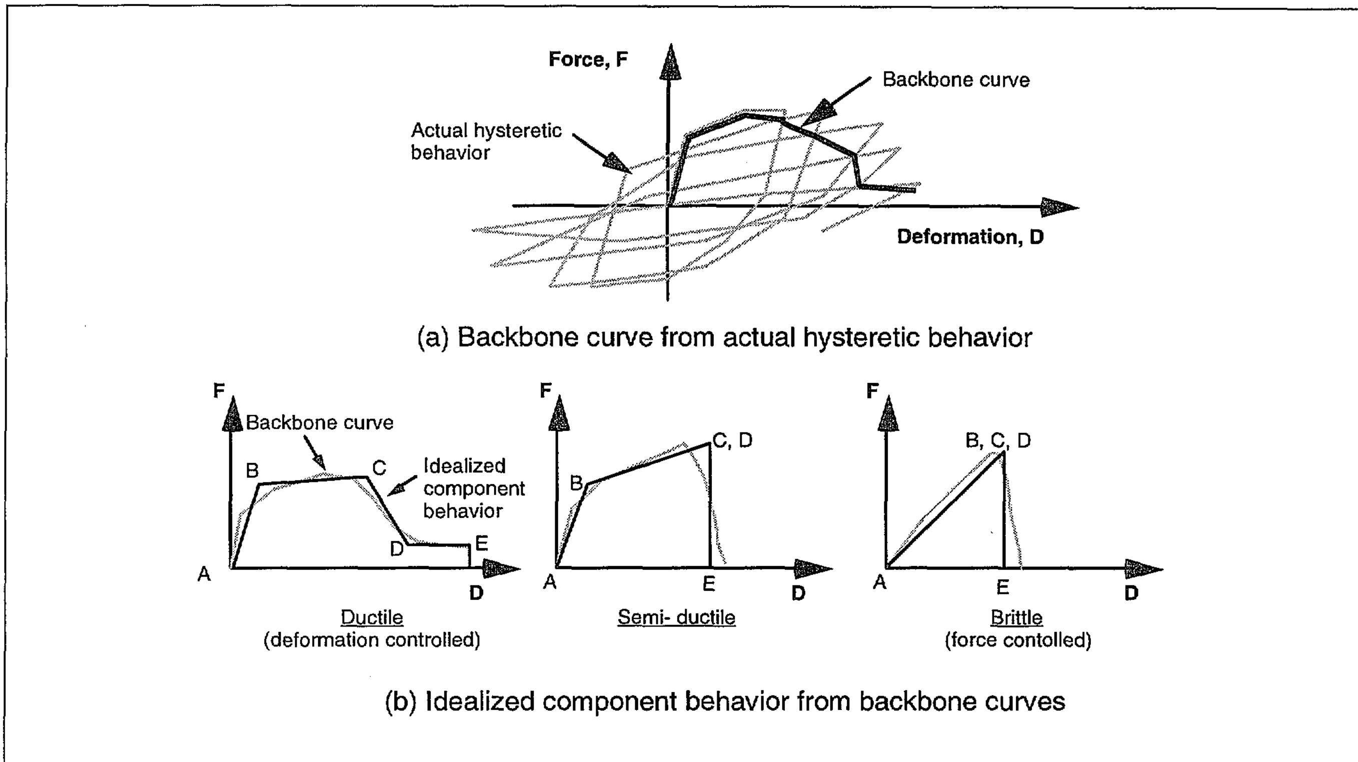


Figure 4-2 ***Idealized Component Force-Deformation Relationship***

Component Damage Classification Guides in Chapters 5 through 8. Using the revised component properties, reformulate the capacity curve for the damaged building and repeat steps 2 and 3 to determine d_c' and d_d'. The ratio of d_c' to d_d' indicates the degree to which the damaged structure satisfies the specified performance objective.

5. If the ratio of d_c' to d_d' is the same, or nearly the same, as the ratio of d_c to d_d, the damage caused by the damaging ground motion has not significantly degraded future performance for the performance objective under consideration.

6. If the ratio of d_c' to d_d' is less than the ratio of d_c to d_d, the effects of the damage caused by the damaging ground motion has diminished the future performance characteristics of the structure. Develop hypothetical actions in accordance with Section 4.5, to restore or augment element and component properties so that the ratio of d_c^* to d_d^* (where the * designates the restored condition) is the same, or nearly the same, as the ratio of d_c to d_d.

4.4.2 Global Displacement Performance Limits

The global displacement performance limits (d_c, d_c', d_c^*) are a function of the acceptability of the deformation of the individual components of the structure as it is subjected to appropriate vertical loads and to a monotonically increasing static lateral load distributed to each floor and roof level in an assumed pattern. The deformation of the components depends on both their geometric configuration in the model and their individual force-deformation chacteristics (see Section 2.4) compared to those of other components. The plot of the total lateral load parameter versus global displacement parameter represents the capacity curve for the building for the assumed load pattern. Thus, the capacity curve is characteristic of the global assembly of individual components and the assumed load pattern.

The current provisions of FEMA 273 limit global displacements for the performance level under consideration (e.g., Immediate Occupancy, Life Safety, Collapse Prevention) to that at which any single component reaches its acceptability limit (see

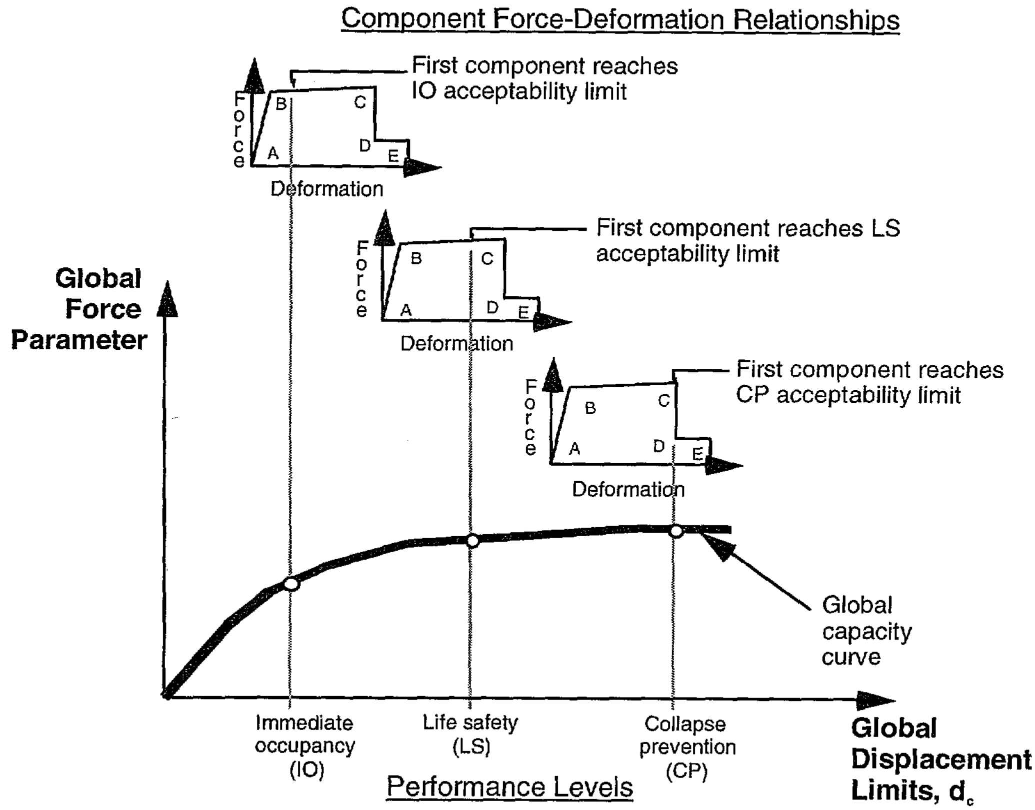


Figure 4-3 ***Global Displacement Limits and Component Acceptability used in FEMA 273/274***

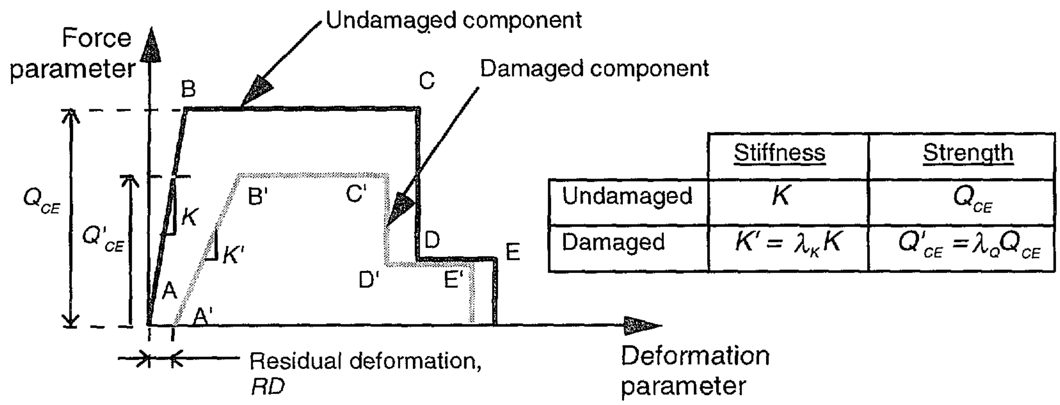
Figure 4-3). The provisions of FEMA 273/274 allow for the re-designation of such components as "secondary". Secondary components have higher deformation acceptability limits but the remaining primary lateral load resisting system components must be capable of meeting acceptability criteria without them. The same allowance may be made for relative performance analysis of earthquake damaged buildings as long as it is applied appropriately to both the pre-event and damaged models.

The acceptability limits were developed for FEMA 273 to identify and mitigate specific seismic deficiencies in buildings to improve anticipated performance. As such, they are intended to be conservative. In a relative performance analysis, the degree of conservatism should be same for both the pre-event and damaged models to give reliable results to estimate the scope of restoration repairs. In an actual earthquake, some "unacceptable" component behavior may not result necessarily in unacceptable global performance. In the future, it is possible that alternative procedures for better estimating global displacement limits will emerge. These also may be suitable for relative performance analyses provided that they are applied consistently and appropriately to both the pre-event and the damaged models.

4.4.3 Component Modeling and Acceptability Criteria

4.4.3.1 Pre-Event Building

In determining the capacity curve for the pre-event building, component properties are generated using the procedures of FEMA 273/274 or ATC-40, modified, if necessary, to reflect the results of the damage



	Stiffness	Strength
Undamaged	K	Q_{CE}
Damaged	$K' = \lambda_K K$	$Q'_{CE} = \lambda_Q Q_{CE}$

Figure 4-4 ***Component Modeling Criteria***

investigation. Modifications may be warranted for two reasons:

1. The procedures assume a normal, relatively minor, degree of deterioration of the building due to service conditions. If the investigation reveals pre-existing conditions (see Section 3.4) that affect component properties beyond these normal conditions, then the "pre-event" component properties must be modified to reflect the condition of the structure just before the earthquake.
2. If the verification process (see Section 3.6) indicates component types or behavior modes inconsistent with the FEMA 273/274 or ATC-40 predicted properties, then the pre-event component properties are modified to reflect the observed conditions.

4.4.3.2 Damaged Building

The effects of damage on component behavior are modeled as shown generically in Figure 4-4. Acceptability criteria for components are illustrated in Figure 4-5. The factors used to modify component properties are defined as follows:

λ_K = modification factor for idealized component force-deformation curve accounting for change in effective initial stiffness resulting from earthquake damage.

λ_Q = modification factor for idealized component force-deformation curve accounting for change in expected strength resulting from earthquake damage.

λ_D = modification factor applied to component deformation acceptability limits accounting for earthquake damage.

RD = absolute value of the residual deformation in a structural component, resulting from earthquake damage.

The values of the modification factors depend on the behavior mode and the severity of damage to the individual component. They are tabulated in the Component Guides in Chapters 5 through 8. The notation λ^* is used to denote modifications to pre-event properties for restored components. These also vary by behavior mode, damage severity, and type of restoration measure, in accordance with the recommendations of Chapters 5 through 8. Figure 4-6 illustrates the general relationship between damage severity and the modification factors. Component stiffness is most sensitive to damage, so this parameter must be modified even when damage is slight. Reduction in strength implies more significant damage. After relatively severe damage, the magnitudes of acceptable displacements are reduced.

4.4.3.3 Establishing λ Factors by Structural Testing

The component modification factors (λ factors) for an earthquake damaged building can be established by

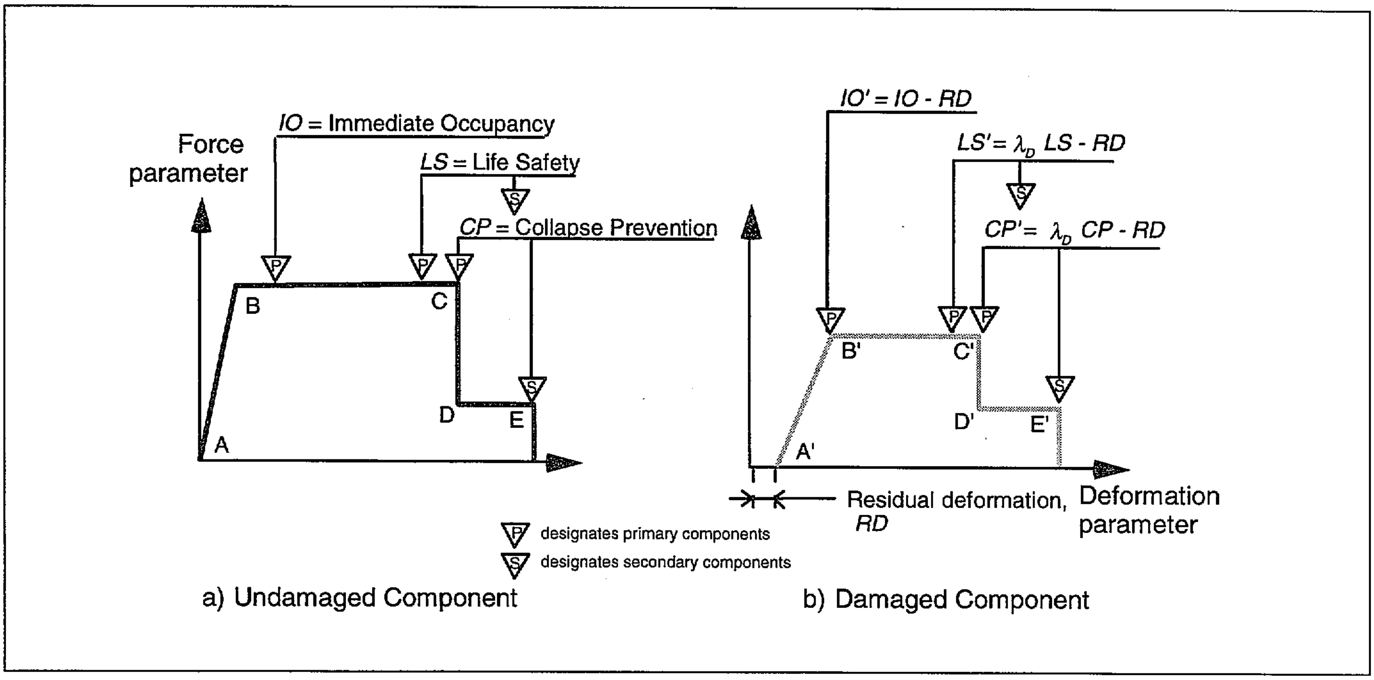


Figure 4-5 Component Acceptability Criteria

laboratory structural testing of critical components, rather than using the values given in the Component Guides of Chapters 5, 6, 7, and 8. The testing must be directly applicable to the specific structural details of the building, and to the damaging and performance earthquakes considered. Typically this would mean that a project-specific test program must be carried out. In certain circumstances, the expense of such a test program may be justified.

If testing is carried out to establish λ values, the test program should conform to the following guidelines:

A. Test Procedure

Two identical test specimens are required for each structural component of interest. One specimen is tested to represent the component in its *post-event* condition subjected to the performance earthquake; the second specimen is tested to represent the component in its *pre-event* condition subjected to the performance earthquake. The λ values are derived from the differences in the force-displacement response between the two specimens.

Figure 4-7 schematically illustrates the required testing. Specimen A is tested first by a load-displacement sequence representative of the damaging earthquake. After this testing, the damage in the specimen should be similar in type and severity to that observed in the actual building after the damaging earthquake.. Specimen A is then tested by a load-displacement sequence representative of the performance level earthquake. From the resulting force-displacement hysteresis data, a backbone curve is drawn, according to Section 2.13.3 of FEMA 273.

A similar testing process is carried out on Specimen B, except that the initial test sequence representing the damaging earthquake is not applied. The λ values are derived from a direct comparison of the backbone curve of Specimen B with the backbone curve of Specimen A.

B. Test Specimens

Each pair of test specimens are to be identical in all details of construction, and in material strengths.

Scale. The scale of the components should be as near to full-scale as is practical. Generally, reinforced concrete specimens should not be tested below 1/4 to 1/3 scale.

Materials. Material strengths for the actual building should be established by testing, and test specimen materials shall be used that match the actual strengths as closely as possible. Material strengths should be identical between Specimen A and Specimen B.

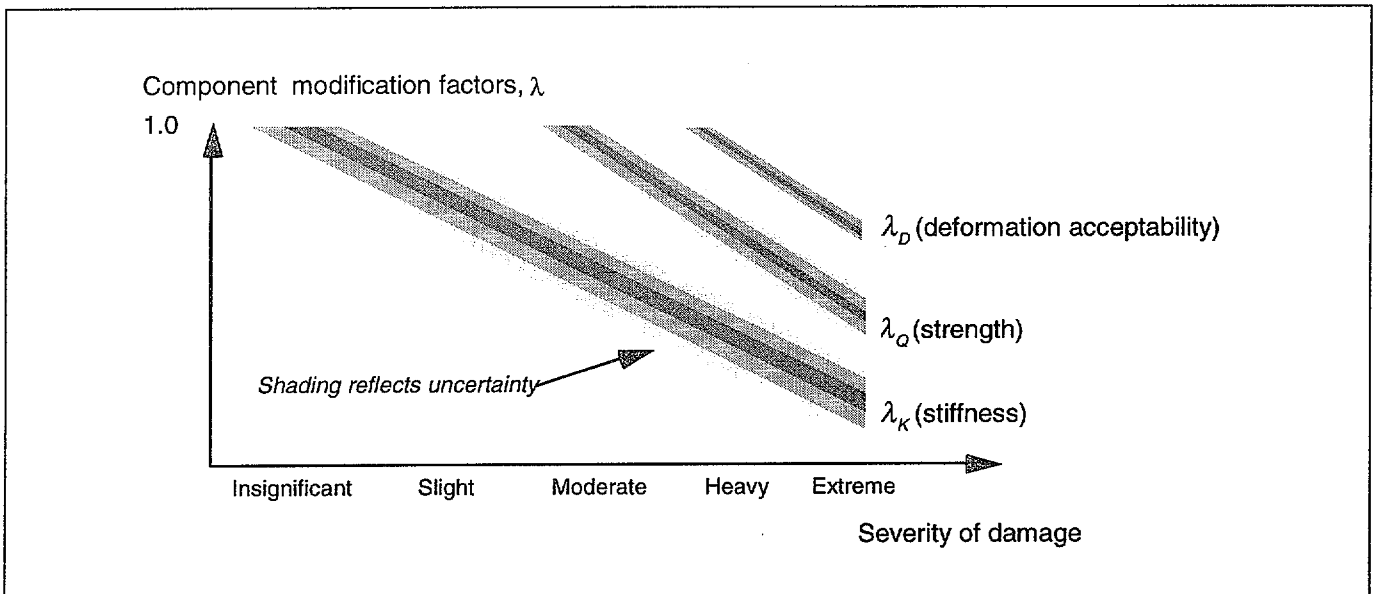


Figure 4-6 ***Component Modification Factors and Damage Severity***

For reinforced concrete structures, it may be preferable to cast Specimens A and B at the same time from the same batch of concrete. Cylinders should be tested to establish the concrete strength at the time each specimen is tested.

Reinforcing steel used in the test specimens should be tested, and the yield strength should be close to that for the actual building. If it is not possible to match yield strength, the area of reinforcement can be adjusted to compensate for the difference in yield strength.

Pre-existing Damage. If the component of the actual building contains damage that is identified as pre-existing (e.g. cracks from shrinkage or a previous earthquake) then, to the extent possible, this damage should be induced in both Specimen A and Specimen B.

Number of Specimens. For simplicity of presentation, these guidelines refer to testing only one pair of specimens. For behavior modes or seismic response that shows substantial variability, more specimens would need to be tested so that results are based on a statistically significant sample.

C. Loading

Cyclic-static loading would typically be used for the testing. The test set-up and applied loading should be designed so that moment and shear diagrams and axial stress levels are representative of those occurring in the actual structure. Established test protocols, e.g. ATC (1992), should be followed to the extent applicable. A pseudo-dynamic loading sequence may be used, with consideration of the issues identified below.

Representing the Damaging Earthquake. To represent the damaging earthquake, the load sequence should recreate the displacement amplitudes and number of cycles undergone by the actual component in the damaging earthquake. Ground motion parameters for the damaging earthquake, from contour maps and recording stations near to the building site, should be reviewed for preliminary estimates of these parameters. The final determination of appropriate displacement amplitudes should be made during testing, so that the loading produces damage that is similar in type and severity to that observed in the actual building.

If a pseudo-dynamic loading sequence is used, it must be designed to allow adjustments during testing to better represent the actual input level of the damaging earthquake.

Representing the Performance Earthquake. To represent the performance earthquake, the load sequence should reflect the demands associated with the selected seismic hazard level. The load sequence should also contain enough cycles at different displacement levels to allow the construction of the backbone curve per Section 2.13.3 of FEMA 273.

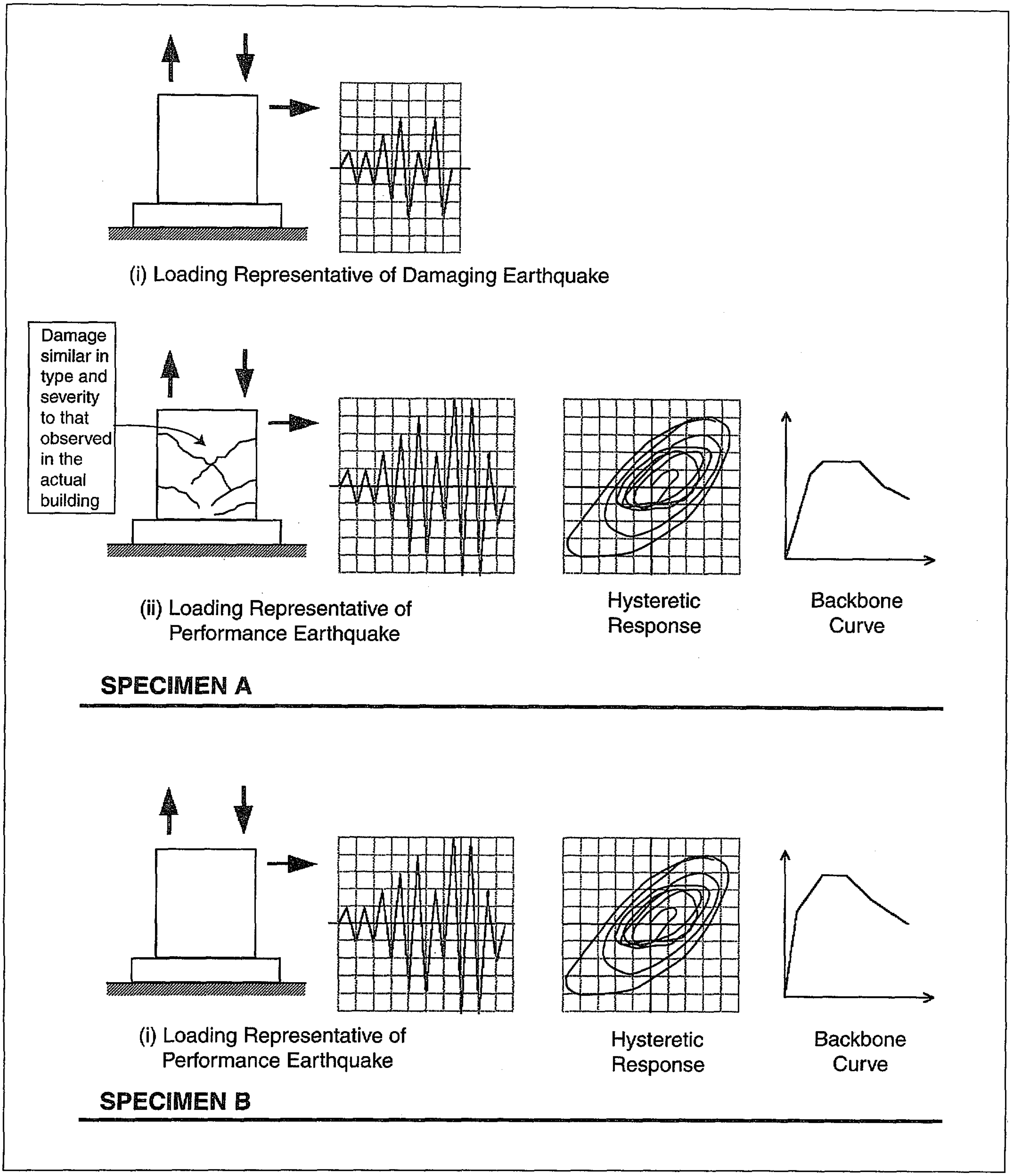


Figure 4-7 *Determining λ values from structural testing*

Pseudo-dynamic test sequences may need to be carefully selected to produce enough cycles in each direction.

4.4.4 Global Displacement Demand

Prior earthquake damage may alter the future seismic response of a building by affecting the displacement demand and the displacement capacity. Effects of prior damage on the future displacement demands may be evaluated according to methods described in this section. Effects of prior damage on displacement capacity are described in Section 4.4.3.

FEMA 307 describes analytical and experimental studies of effects of prior damage on future earthquake response demands. A primary conclusion is that prior earthquake damage often does not cause a statistically significant change in maximum displacement demand for the overall structural system in future earthquakes under the following circumstances:

a. there is not rapid degradation of resistance with repeated cycles.

b. the performance ground motion associated with the future event produces a maximum displacement, d_d, larger than that produced by the damaging ground motion, d_e.

c. the residual drift of the damaged or repaired structure is small relative to d_e.

If the performance ground motion produces a maximum displacement, d_d' less than that produced by the damaging ground motion, d_e. the response of the damaged structure is more likely to differ from that of the pre-event structure, d_d (see Figure 4-8).

There are several alternatives for estimating the displacement demand for a given earthquake motion. FEMA 273 relies primarily on the displacement coefficient method. This approach uses a series of coefficients to modify the hypothetical linear-elastic response of a building to estimate its nonlinear-inelastic displacement demand. The capacity spectrum method (ATC 40) characterizes seismic demand initially using a 5% damped linear-elastic response spectrum and reduces the spectrum to reflect the effects of energy dissipation in an iterative process to estimate the inelastic displacement demand. The secant stiffness method (Kariotis et al., 1994), although formatted differently, is fundamentally similar to the capacity spectrum method. Both these latter two methods can be related to the substitute structure method (Shibata and Sozen, 1976). The use of each of these approaches to generate estimates of global displacement demand (d_d, $d_{d'}$, and d_{d*}) is summarized in the following sections. Generally, any of the methods may be used for the evaluation of the effects of damage; however, the same method should be used to calculate each of the global displacement demands (d_d, $d_{d'}$, and d_{d*}) when making relative comparisons using these parameters.

4.4.4.1 Displacement Coefficient Method

The displacement coefficient method refers to the nonlinear static procedure described in Chapter 3 of FEMA 273. The method also is described in Section 8.2.2.2 of ATC 40. The reader is referred to those documents for details in application of the procedure. A general overview and a description of the application of the method to damaged buildings are presented below.

The displacement coefficient method estimates the earthquake displacement demand for the building using a linear-elastic response spectrum. The response spectrum is plotted for a fixed value of equivalent damping, and the spectral response acceleration, S_a, is read from the spectrum for a period equal to the effective period, T_e. The effective period is defined by the following:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (4\text{-}1)$$

where T_i is the elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis, K_i is the elastic lateral stiffness of the building in the direction under consideration (refer to Figure 4-9), and K_e is the effective lateral stiffness of the building in the direction under consideration (refer to Figure 4-9). As described in FEMA 273, the effective lateral stiffness is taken as a secant to the capacity curve at base shear equal to $0.6V_y$. For a concrete or masonry wall building that has not been damaged previously by an earthquake, the effective damping is taken equal to 5% of critical damping.

The target displacement, δ_t, is calculated as:

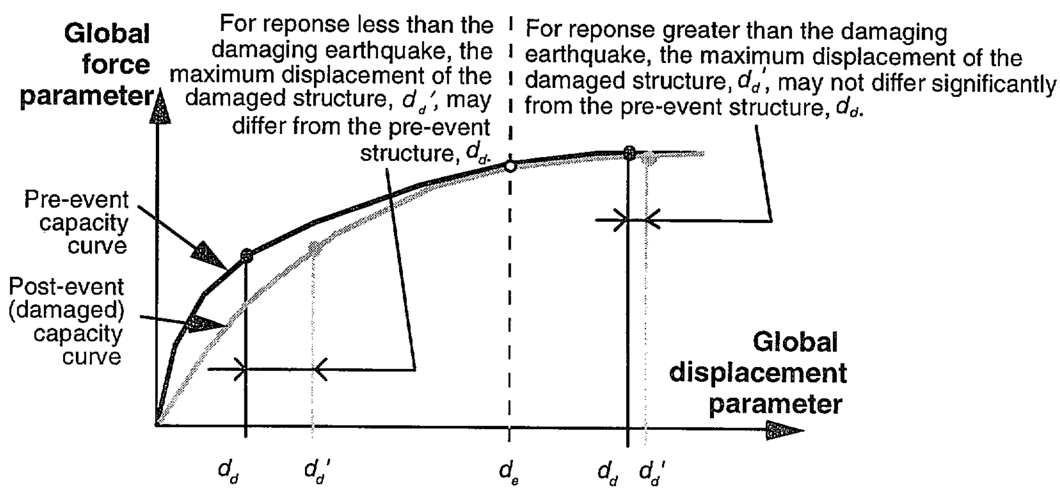


Figure 4-8 ***Maximum Displacement Dependency on Damaging Earthquake***

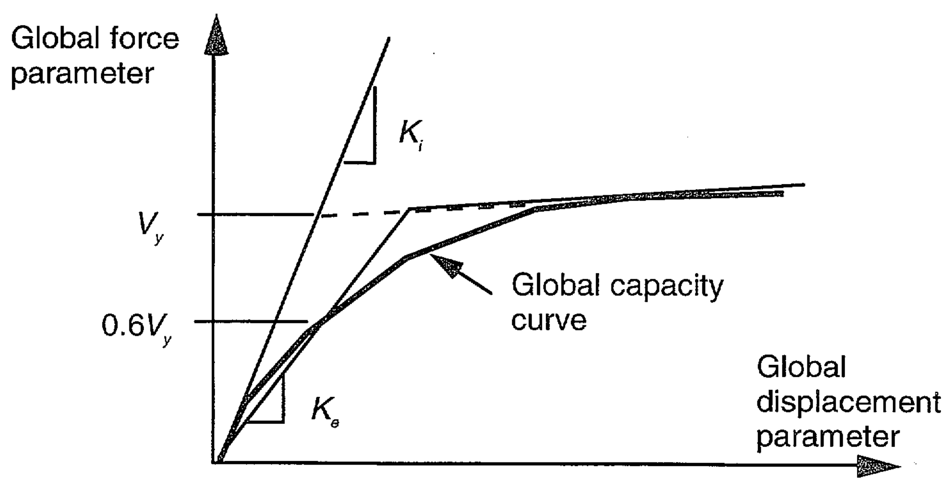


Figure 4-9 ***Global Capacity Dependency on Initial and Effective Stiffness***

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \qquad (4\text{-}2)$$

where C_0, C_1, C_2, C_3 are modification factors defined in FEMA 273, and all other terms are as defined previously.

The maximum displacement, d_d, of the building in its pre-event condition for a performance ground motion is estimated by applying the displacement coefficient method using component properties representative of the pre-event conditions. To use the displacement coefficient method to estimate the maximum displacement demand, d_d', during a performance

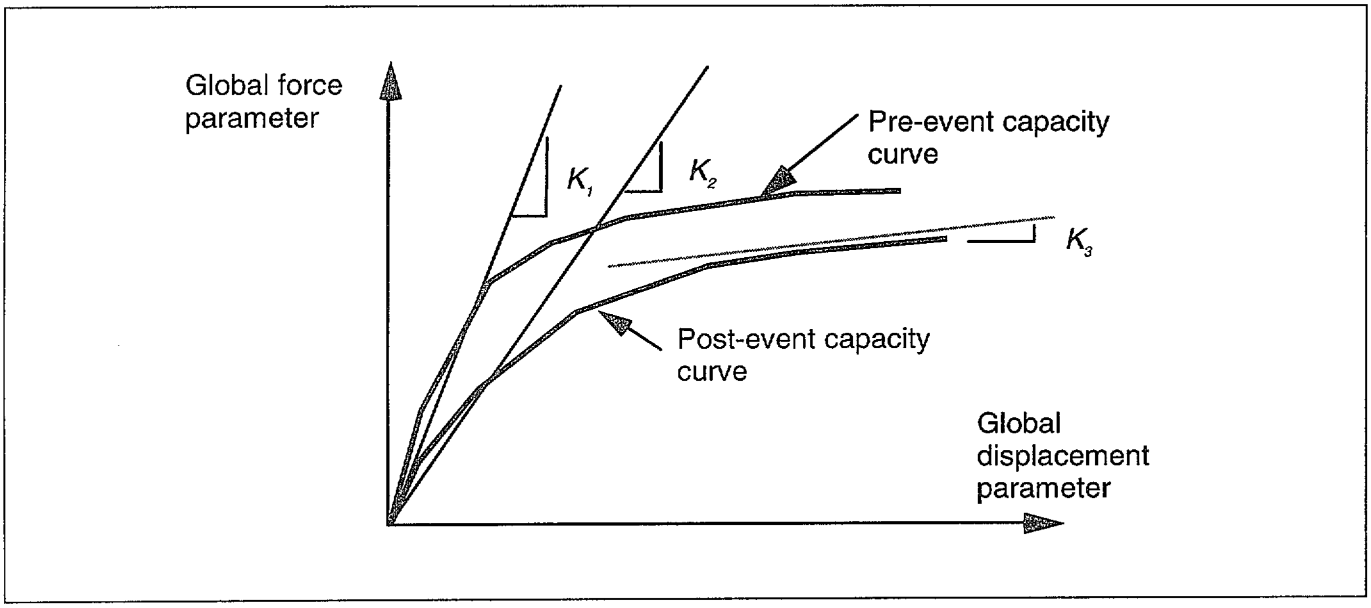


Figure 4-10 ***Pre- and Post-Event Capacity Curves with Associated Stiffnesses***

ground motion for a building damaged by a previous earthquake use the following steps: (See Figure 4-10.)

1. Construct the relation between lateral seismic force (base shear) and global structural displacement (roof displacement) for the pre-event structure. Refer to this curve as the pre-event capacity curve. Pre-event force-displacement relations should reflect response characteristics observed in the damaging earthquake, as discussed in Section 3.6.
2. Construct a similar relationship between lateral seismic force and global structural displacement for the structure based on the damaged condition of the structure, using component modeling parameters defined in Section 4.4.3. Refer to this curve as the post-event capacity curve.
3. Define effective stiffnesses K_1, K_2, and K_3 as shown in Figure 4-10. K_1 is K_e (see Figure 4-9) calculated from the pre-event capacity curve. K_2 is K_e (see Figure 4-9) calculated from the post-event capacity curve. K_3 is the effective post-yield stiffness from the post-event capacity curve.
4. Apply the displacement coefficient method as defined in FEMA 273 with the effective stiffness taken as $K_e = K_1$, effective damping equal to 5% of critical damping, post-yield stiffness defined by stiffness K_3, and effective yield strength defined by the intersection of the lines having slopes K_1 and K_3 to calculate δ_t using Equation 4-2. Assign the displacement parameter d'_{d1} the value calculated for δ_t.
5. Apply the displacement coefficient method as defined in FEMA 273 with the effective stiffness taken as $K_e = K_2$, effective damping as defined by Equation 4-3, post-yield stiffness defined by stiffness K_3, and effective yield strength defined by the intersection of the lines having slopes K_2 and K_3 to calculate the displacement parameter d'_{d2}.
6. Using the displacement parameters d'_{d1} and d'_{d2}, estimate the displacement demand, d'_d, for the structure in its damaged condition as follows:
   a. If d'_{d1} is greater than d_e, then $d'_d = d'_{d1}$
   b. If d'_{d1} is less than d_e, then $d'_d = d'_{d2}$

The effective damping as defined by Equation 4-3 is consistent with experimental results obtained by Gulkan and Sozen (1974),

$$\beta = 0.05 + 0.2\left[1 - \left(\frac{K_2}{K_1}\right)^{0.5}\right] \qquad (4\text{-}3)$$

For a restored or upgraded structure, the displacement demand, d^*_d, for a performance ground motion may be calculated using the displacement coefficient method with 5% damping using a capacity curve generated using applicable properties for existing components, whether repaired or not, and any supplemental components added to restore or upgrade the structure.

4.4.4.2 Capacity Spectrum Method

The capacity spectrum method is described in Section 8.2.2.1 of ATC 40. The reader is referred to that document for details in application of the procedure. A general overview and a description of the application of the method to damaged buildings are presented below.

The capacity spectrum method estimates the earthquake displacement demand for the building using a linear-elastic response spectrum. The response spectrum is plotted for a value of equivalent damping based on the degree of nonlinear response, and the spectral displacement response is read from the intersection of the capacity curve and the demand curve. In some instances of relatively large ground motion, the curves may not intersect, indicating potential collapse. In these cases the displacement coefficient method could be used as an alternate method for damage evaluation.

The maximum displacement of the building in its pre-event condition, d_d, for a performance ground motion is estimated by applying the capacity spectrum method using component properties representative of the pre-event conditions. To use the capacity spectrum method to estimate the maximum displacement demand, d'_d, during a performance ground motion for a building damaged by a previous earthquake, use the following steps:

1. Construct the relation between lateral seismic force (spectral acceleration) and global structural displacement (spectral displacement) for the structure assuming the damaging ground motion and its resultant damage had not occurred. Pre-event component force-deformation relationships should reflect response characteristics observed in the damaging earthquake as discussed in Section 3.6. Refer to this curve as the pre-event capacity curve.
2. Construct a similar relation between lateral seismic force and global structural displacement for the structure based on the damaged condition of the structure, using component modeling parameters defined in Section 4.4.3. Refer to this curve as the post-event capacity curve.
3. Apply the capacity spectrum method using the pre-event capacity curve to calculate the displacement parameter d'_{d1}.
4. Apply the capacity spectrum method using the post-event capacity curve to calculate the displacement parameter d'_{d2}. For determining the effective damping, the yield strength and displacement for the post-event capacity curve should be taken identically equal to the yield strength and displacement determined for the pre-event capacity curve. (See Equation 4-3.)
5. Using the displacement parameters d'_{d1} and d'_{d2}, estimate the displacement demand, d'_d, for the structure in its damaged condition as follows:
   a. If d'_{d1} is greater than d_e, then $d'_d = d'_{d1}$
   b. If d'_{d1} is less than d_e, then $d'_d = d'_{d2}$

For a restored or upgraded structure the displacement demand for a performance ground motion, d_{d*}, may be calculated using the capacity spectrum method based on a capacity curve using applicable properties for existing components, whether repaired or not, and any supplemental components added to restore or upgrade the structure.

4.4.4.3 Secant Stiffness Method

The secant stiffness method is described in Section 8.4.2.1 of ATC-40, *Seismic Evaluation and Retrofit of Concrete Buildings* (ATC, 1996). The reader is referred to that document for details in application of the procedure. To use the method for damaged buildings, the general procedure should be applied based on the properties of the damaged building.

4.4.4.4 Nonlinear Dynamic Procedure

As an alternative to the nonlinear static procedures described above, nonlinear dynamic response histories may be computed to estimate the displacement demand for the building. This dynamic analysis approach requires that suitable ground motion records be selected for both the damaging event and the performance ground motion. It also requires that representative structural models be prepared for the building in its pre-event (no superscript), damaged ('), and restored or upgraded (*) conditions. Detailed procedures have not been developed for the use of nonlinear dynamic response histories in relative performance analyses. The following sections offer general guidance on nonlinear dynamic procedures consistent with the nonlinear static procedures.

A. Ground Motions

Damaging Event. If available, ground motion time histories at, or near, the site may be used to represent the damaging ground motion. Alternatively, an estimate of spectral response can be generated using the procedures of Section 3.1. Time histories consistent with the estimated spectral response may then be generated to represent the damaging ground motion. The average maximum displacement response, d_d, of the pre-event structural model to the time histories should be near that which is estimated to have actually occurred in the structure under evaluation. This effort may involve some adjustments to both the structural model and the ground motion in a verification process, similar to that outlined in Section 3.6, to calibrate the analysis with the observed damage.

Performance Ground Motions. Three to five ground motion accelerograms might be used to represent potential motions at the site for each performance level considered. Each of the records should be consistent with the response spectra that would be used with the nonlinear static procedures for the performance level under consideration as presented in FEMA 273/274 (ATC, 1997a,b) and ATC-40 (ATC, 1996). The maximum global displacement responses of the structural models can be averaged to generate a best estimate of the response.

B. Structural Modeling

The analysis procedure will vary, depending on the type of model used for the individual structural components.

Non-degrading Component Models. If the components are modeled using force-deformation relations that do not include strength degradation (non-degrading model), a procedure to estimate displacement demands is as follows:

1. Determine the maximum displacement response of the pre-event building, d_d, to the performance ground motion using a structural model with component properties representative of pre-event conditions. This pre-event model should be calibrated, as discussed above, to the damaging ground motion.
2. Modify the component properties to reflect the effects of the observed damage in accordance with the recommendations of Section 4.4.3. Determine the maximum displacement response of the building in its damaged condition, d_d', to the performance ground motion using the damaged structural model.
3. Modify the model to reflect the effects of restoration or upgrade measures. These may include the modification of existing components or the addition of new components as discussed in Section 4.5. Determine the maximum displacement response of the building in its restored or upgraded condition, d_d^*, to the performance ground motion using the restored or upgraded structural model.

Degrading Component Models. If the components are modeled using force-deformation relations that allow strength degradation during the response history (degrading model), a procedure to estimate displacement demands is as follows:

1. Determine the maximum displacement response of the pre-event building, d_d, to the performance ground motion using a structural model with component properties representative of pre-event conditions. This pre-event model should be calibrated, as discussed in Section 3.6, to the damaging ground motion.
2. Subject the pre-event model to a composite ground motion comprised of the damaging ground motion, followed by a quiescent period in which the structure comes to rest, followed by the performance ground motion. Estimate the maximum displacement response, d_d', of the building in its damaged condition as the maximum displacement to occur in the time period after the quiescent period in the record.
3. Modify the model to reflect the effects of both the damage and restoration or upgrade measures. These may include the modification of existing components or the addition of new components as discussed in Section 4.5. Determine the maximum displacement response, d_d^*, of the building in its restored or upgraded condition to the performance ground motion using the modified structural model.

4.5 Performance Restoration Measures

If the performance capability of the structure is diminished by the effects of earthquake damage $(d_c' / d_d' < d_c / d_d)$, the magnitude of the loss is quantified by the costs of performance restoration measures. These are hypothetical actions that, if implemented, would result in future performance approximately equivalent to the undamaged building ($d_c^* / d_d^* \approx d_c / d_d$). Performance restoration measures may take several different forms:

- Component restoration entails the repair of individual components to restore structural properties that were diminished as a result of the earthquake damage. The Component Guides in Chapters 5 through 8 provide guidance based on component type, behavior mode, and severity of damage. They refer to outline specifications for individual restoration techniques that are compiled in FEMA 308: *The Repair of Earthquake Damaged Concrete and Masonry Wall Buildings*.

- An extreme case of component repair is complete replacement. In some cases, this is the only alternative indicated in the Component Guides. In other cases, it may be a more economical alternative than component repair.

- Performance can also be restored by the addition of new supplemental lateral-force-resisting elements or components.

Performance restoration measures are specified at the component level using one or more of the above alternatives. The measures are then tested by analyzing the performance of the modified structure, as outlined in Section 4.4. If necessary, the scope of the measures should be adjusted until the performance is approximately the same as that of the undamaged building. It should be noted that all components need not necessarily be restored individually to restore overall performance. It is advisable to explore several strategies to reach an economical solution.

Once the scope has been determined, the loss associated with the earthquake damage can be calculated as the cost of the performance restoration measures if they were to be actually implemented. The cost should include estimates of direct construction costs, as well as the associated indirect costs. Indirect costs include project costs such as design and management fees. In some cases, the costs of hypothetical temporary relocation of building occupants, loss of revenue, and other indirect costs, should be included. Detailed guidelines on determining both direct and indirect costs are not included in this document.

4.6 An Alternative—The Direct Method

A direct method of determining performance restoration measures may be used to estimate the loss caused by the earthquake. This method assumes that the scope of performance restoration measures is equivalent to restoring the significant structural properties of all of the components of the structure. The method uses individual repair actions for each component addressing the observed damage directly without explicitly considering its effect on seismic performance. These repair actions are summarized in the Component Guides of Chapters 5 through 8. The fundamental assumption is that the restored structure would have equivalent performance capability to the undamaged building for performance ground motions greater than that of the damaging ground motion ($d_d^* > d_e$). For this case the direct method will tend to overestimate losses, because some of the damage will have no effect on seismic performance. If the anticipated performance ground motion is less than that of the damaging ground motion ($d_d^* < d_e$), the direct method may underestimate losses in some cases, depending on individual building characteristics. This is because it neglects some loss of stiffness in components that theoretically could increase displacement response for smaller events. In these cases, relative performance analysis may be necessary to evaluate losses more accurately.

The direct method should be used only if the sole objective is to estimate the loss from the damaging earthquake based on the cost of the performance restoration measures. The direct method provides no information on the actual performance of the building in its damaged or undamaged states and cannot be used for design purposes.

Relative performance analysis is preferred because it determines the seismic performance of the building in its damaged and undamaged states.The scope of performance restoration measures is determined by analyzing their effect on the predicted performance. Since the effect on global performance of damage to individual components is considered, this technique generally provides a more accurate evaluation of the actual loss due to the damaging earthquake. The analysis also provides information that may be used for design purposes to restore or upgrade the building.

5: Reinforced Concrete

5.1 Introduction and Background

This section provides information on reinforced concrete wall components including the Component Damage Classification Guides (Component Guides) for reinforced concrete (RC). Section 5.2 defines and describes the component types and behavior modes that may be encountered in reinforced concrete wall structures. Section 5.3 gives evaluation procedures for assessing component strength and determining likely behavior modes. Section 5.4 defines symbols for studies on reinforced concrete walls.

The Component Guides and evaluation procedures in Section 5.5 are based on a review of the applicable research. Extensive structural testing has been done on reinforced concrete walls and wall components. In-plane tests on concrete walls in the 1950s and early 1960s generally applied monotonic loading and focused on ultimate strength without considering displacement or ductility capacity. Two major programs during this time period were carried out at Stanford University (Benjamin and Williams, 1957 and 1958) and at MIT (Antebi et al., 1960).

From the late 1960s to early 1980s, the Portland Cement Association conducted a comprehensive and pioneering test program on the earthquake resistance of reinforced concrete walls (Cardenas, 1973; Oesterle et al., 1976, 1979, 1983; Shiu et al., 1981; Corley et al., 1981). The tests principally used cyclic-static loading and identified several possible behavior modes in reinforced concrete walls, including flexural behavior, diagonal tension, diagonal compression (web crushing), boundary compression and bar buckling, sliding shear, and out-of-plane wall buckling.

Research on the behavior of coupling beams of walls was begun in the late 1960s at the University of Canterbury in New Zealand (Paulay, 1971a, b; Paulay & Binney, 1974; Paulay and Santhakumar, 1976). Research on numerous aspects of reinforced concrete wall behavior continued at Canterbury through the 1990s, covering ductile flexural behavior, diagonal tension, boundary confinement and bar buckling, sliding shear, out-of-plane stability, and walls with irregular openings (Paulay, 1980, 1986; Paulay et al., 1982; Paulay & Priestley, 1992, 1993). The research findings have contributed to code provisions for reinforced concrete walls in the United States and elsewhere.

As part of a U.S.- Japan cooperative research program, a seven-story full-scale, shear-wall structure was tested (Wight, 1985). This test demonstrated the contribution made by beams and slabs that framed into the wall to the capacity of wall structures.

Wall testing has also been carried out at the University of California, Berkeley (Wang et al., 1975; Vallenas et al., 1979; Iliya & Bertero, 1980) and on small shake-table specimens at the University of Illinois (Aristisabal and Sozen, 1976; Lybas and Sozen, 1977). More recent tests have been done by Wallace and Thomsen (1995). A number of tests have been carried out in Japan, including those by Ogata and Kabeyasawa (1984).

5.2 Reinforced Concrete Component Types and Behavior Modes

5.2.1 Component Types

Five possible component types are defined for reinforced concrete wall structures. The component types are listed and described in Table 5-1. Typically only component types RC1, RC2, and RC3 will suffer earthquake damage. Component types RC4 and RC5 are mentioned for completeness, but since they are not expected to suffer earthquake damage, they are not discussed in detail.

Wall Component types are assigned based on identifying the governing mechanism for nonlinear lateral deformation for the structure, as described in Section 2.4.

5.2.2 Behavior Modes and Damage

The possible modes of nonlinear behavior and damage for reinforced concrete wall components are outlined in Table 5-2, along with their response characteristics. Section 2.2 of FEMA 307 presents typical force-displacement hysteresis loop shapes for the behavior modes. The likelihood of such behavior modes occurring in each of the prevalent component types is shown in Table 5-3.

Table 5-1 Component Types and Descriptions for Reinforced Concrete Walls.

Component Type		Description
RC1	Isolated Wall or Stronger Wall Pier	Stronger than beam or spandrel elements that may frame into it, so that nonlinear behavior (and damage) is generally concentrated at the base, with, for example, a flexural plastic hinge or shear failure. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior.
RC2	Weaker Wall Pier	Weaker than the spandrels to which it connects, characterized, for example, by flexural hinging at top and bottom, or shear failure.
RC3	Weaker Spandrel or Coupling Beam	Weaker than the wall piers to which it connects, characterized, for example, by hinging at each end, shear failure, or sliding shear failure.
RC4	Stronger Spandrel	Should not suffer damage because it is stronger than attached piers. If this component is damaged, it should probably be re-classified as RC3.
RC5	Pier-Spandrel Panel Zone	Typically not a critical area in RC walls.

The behavior modes are described in the following sections according to the ductility categories given in Table 5-2.

This document focuses on structures for which earthquake damage occurs primarily in wall components. Engineers should be aware that damage can also occur in other structural elements such as foundations, columns, beams, and slabs.

5.2.3 Behavior Modes with High Ductility Capacity (Flexural Response)

Adequately designed reinforced concrete walls of various configurations can respond to earthquake shaking in a ductile manner. Ductile wall response usually results from flexural behavior, which requires that the wall components be designed to avoid the following less desirable behavior effects:

- Failures in shear corresponding to diagonal tension, web crushing, or sliding shear
- Buckling of longitudinal bars in boundary regions of plastic hinge zones
- Loss of concrete strength due to high compressive strains in unconfined boundary regions of plastic hinge zones
- Slip of lap splices
- Out-of-plane buckling of thin wall sections

The strength of wall components in flexure is calculated using conventional procedures given in Section 5.3.5. Wall components responding in flexure generally have good displacement capacity, typically in-plane rotations exceeding two percent (or 0.02 radians), or displacements at least eight times yield.

5.2.4 Behavior Modes with Intermediate Ductility Capacity

The following behavior modes can be defined as having intermediate ductility capacity:

- Flexure/Diagonal tension
- Flexure/Web crushing
- Flexure/Sliding shear
- Flexure/Boundary-zone compression
- Flexure/Lap-splice slip
- Flexure/Out-of-plane wall buckling

The earthquake response in these behavior modes is initially governed by flexure, but after some number of cycles, reaching some level of earthquake displacements, a response mode other than flexure predominates. At this point, the component suffers strength degradation.

Table 5-2 ***Behavior Modes for Reinforced Concrete Wall Components.***

Behavior Mode	Approach to calculate strength (use expected material values)	Approach to estimate displacement capacity	Ductility Category
A. Ductile flexural response	Conventional calculations per Section 5.3.5.	Good displacement capacity (e.g. 2% drift, or 8x yield displacement).	High ductility capacity
B. Flexure/ Diagonal tension	Moment strength per Section 5.3.5 initially governs strength.	Based on shear strength as a function of ductility. See Section 5.3.6.b.	Ductility capacity varies (Failure only occurs after some degree of flexural yielding and concrete degradation.)
C. Flexure/ Diagonal compression (web crushing)		Based on relationship of web crushing strength to drift per Oesterle et al (1983). See Section 5.3.6.c.	
D. Flexure/ Sliding shear		Shear friction approach per ACI 318, or recommendations of Paulay and Priestley (1992). See Section 5.3.6.d.	
E. Flexure/ Boundary-zone compression		Based on amount of ties required for moderate and high ductility levels, per Paulay and Priestley (1992). See Section 5.3.7.	
F. Flexure/Lap-splice slip		Based on lap strength as a function of ductility. See Section 5.3.8	
G. Flexure/Out-of-plane wall buckling		Based on wall thickness requirements for moderate and high ductility levels. See Section 5.3.9.	
H. Preemptive diagonal tension	Shear strength governs at low ductility levels, per Section 5.3.6.b.	No inelastic displacement capacity.	Little or no ductility capacity (Flexural reinforcement does not yield.)
I. Preemptive web crushing	May occur at shear stresses of $12\sqrt{f'_{ce}}$ – $15\sqrt{f'_{ce}}$. See Section 5.3.6.c.		
J. Preemptive sliding shear	Shear friction approach per ACI 318. See Section 5.3.6.d.		
K. Preemptive boundary zone compression	Applies only to unusually high axial loads, above the balance point. Moment strength calculation still governs.		
L. Preemptive lap-splice slip	Lap strength, per FEMA 273 and ATC-40, or approach of Priestly et al. (1996) governs. See Section 5.3.8.		
M. Global foundation rocking of wall	See FEMA 273 or ATC-40		Moderate to high ductility capacity
N. Foundation rocking of individual piers			

Table 5-3 ***Likelihood of Earthquake Damage to Reinforced Concrete Walls According to Wall Component and Behavior Mode.***

Behavior Mode	**Wall Component Type**		
	Isolated Wall or Stronger Wall Pier (RC1)	**Weaker Wall Pier** (RC2)	**Weaker Spandrel or Coupling Beam** (RC3)
A. Ductile flexural response	Common in well-designed walls See Guide RC1A	May occur See Guide RC2A	May occur, particularly if diagonally reinforced Similar to Guide RC2A
B. Flexure/Diagonal tension	Common See Guide RC1B	Common Similar to Guide RC1B	Common See Guide RC3B
C. Flexure/Diagonal compression (web crushing)	Common (frequently observed in laboratory tests) See Guide RC1C	May occur	May occur
D. Flexure/Sliding shear	May occur, particularly for squat walls See Guide RC1D	May occur	Common See Guide RC3D
E. Flexure/ Boundary-zone compression	Common See Guide RC1E	May occur	Unlikely
F. Flexure/Lap-splice slip	May occur	May occur	May occur
G. Flexure/Out-of-plane wall buckling	May occur (observed in laboratory tests)	Unlikely	Unlikely
H. Preemptive diagonal tension	Common Similar to Guide RC2H	Common See Guide RC2H	Common Similar to Guide RC2H
I. Preemptive web crushing	May occur in squat walls (observed in laboratory tests)	May occur	May occur
J. Preemptive sliding shear	May occur in very squat walls or at poor construction joints.	May occur in very squat walls or at poor construction joints.	Unlikely
K. Preemptive boundary zone compression	May occur in walls with unsymmetric sections and high axial loads	May occur in walls with unusually high axial load	Unlikely
L. Preemptive lap-splice slip	May occur	May occur	May occur
M. Global foundation rocking of wall	Common	n/a	n/a
N. Foundation rocking of individual piers	May occur	May occur	n/a

Notes:

- Shaded areas of table indicate behavior modes for which a specific Component Damage Classification Guide is provided in Section 5.5. The notation *Similar to Guide...* indicates that the behavior mode can be assessed by using the guide for a different, but similar, component type or behavior mode.
- *Common* indicates that the behavior mode has been evident in postearthquake field observations and/or that experimental evidence supports a high likelihood of occurrence.
- *May occur* indicates that the behavior mode has a theoretical or experimental basis, but that it has not been frequently reported in postearthquake field observations.
- *Unlikely* indicates that the behavior mode has not been observed in either the field or the laboratory.

a. Strength and Displacement Capacity

Flexural behavior governs the maximum strength achieved in behavior modes with intermediate ductility capacity. For these behavior modes, the full flexural strength will not be sustained at high levels of cyclic deformation. Section 5.3.5 gives guidelines for calculating flexural strength.

Displacement capacity may be difficult to assess for behavior modes with intermediate ductility capacity. One approach for estimating displacement capacity is to consider the intersection of the force-displacement curve for flexural response with a degrading strength envelope for the governing failure mechanism. The degrading strength envelope may represent, for example, lap-splice strength or shear strength. Useful research has been carried out using this approach. For example, Priestley et al. (1996) have developed specific recommendations on the degradation of strength as a function of ductility for lap-splice failure and shear failure.

b. Flexure/Diagonal Tension

The flexure/diagonal tension behavior mode occurs in a wall component when the shear strength in diagonal tension initially exceeds the flexural strength, allowing flexural yielding to occur. However, after the cracks open and the concrete in the plastic hinge zone degrades, the shear strength is reduced below the flexural strength, and shear behavior predominates.

At low levels of response, this behavior mode may appear similar to a ductile flexural response, although diagonal cracks due to shear stress may be more prominent. At higher levels of response, diagonal cracking tends to concentrate in one or two wide cracks. Eventually horizontal reinforcement can be strained to the point of fracture, signaling a diagonal tension failure.

c. Flexure/Diagonal Compression (Web Crushing)

For heavily reinforced walls subject to high shear forces, shear-related compression failures may occur rather than diagonal tension failures. This mode of behavior has been commonly observed in laboratory testing, and it may be prevalent in low-rise walls or when shear reinforcement is sufficient to prevent a diagonal tension failure. Higher axial loads also increase the likelihood of web-crushing behavior.

Web crushing generally occurs after some degree of cyclic flexural behavior and degradation. The vulnerability to web crushing can be considered to be proportional to the story drift ratio to which the component is subjected. This behavior mode is characterized by diagonal cracking and spalling in the web region of the wall. Localized web crushing can be initiated by the uneven closing of diagonal cracks under cyclic earthquake forces.

d. Flexure/Sliding Shear

Coupling beams and low-rise walls are particularly vulnerable to failure by sliding shear. Low axial loads and poor construction joint details increase the probability of sliding shear.

In this behavior mode, flexural yielding initially governs the response. Flexural cracks at the critical section tend to join up to form a single crack across the section which becomes a potential sliding plane. Under cyclic forces and displacements, this crack opens more widely so that the aggregate interlock and shear friction resistance on the sliding plane degrade. When the sliding shear strength drops below the shear corresponding to the moment strength, lateral sliding offsets begin to occur.

For many low-rise walls, lateral strength may be governed by the strength of the foundation to resist overturning. Sliding shear behavior is likely to occur only in low-rise walls where the foundations have the capacity to force flexural yielding.

e. Flexure/Boundary-Zone Compression

Taller walls with adequate shear strength but inadequate boundary tie reinforcement tend to be vulnerable to this behavior mode. Under inelastic flexural response, the boundary regions of plastic hinge zones may be subjected to high compression strains, which cause spalling of the cover concrete. If sufficient tie reinforcement is not placed around the longitudinal bars in the wall boundaries, the longitudinal bars are prone to buckling. Additionally, in walls where concrete compressive strains exceed 0.004 or 0.005, the concrete in the boundary regions can rapidly lose compressive strength if it is not confined by adequate boundary ties. In addition to bar-buckling restraint and confinement, ties around the lap splices of boundary longitudinal bars significantly increase lap-splice strength.

f. Flexure/Lap-Splice Slip

Lap splices in the critical plastic hinge regions of walls are commonly encountered in existing buildings. Even when relatively good lap-splice length is provided, lap splices in plastic hinge zones tend to slip when the concrete compressive strain exceeds 0.002, unless ties are provided around the lap splices.

Slipping of lap splices is accompanied by splitting cracks in the concrete, oriented parallel to the spliced reinforcement. The use of tie reinforcement around lap splices, which restrains the opening of the splitting cracks, can prevent or delay the onset of lap-splice slip.

Once lap splices slip, the component strength falls below the full moment strength of the section and the strength is governed by the residual strength of the splices plus the moment capacity due to axial load.

g. Flexure/Out-of-Plane Wall Buckling

Several experimental studies have shown that thin wall sections can experience out-of-plane buckling when subjected to cyclic flexural forces and displacements. For typical wall sections the buckling occurs only at high ductility levels.

Single curtain walls and walls with higher amounts of longitudinal reinforcement tend to be more vulnerable to out-of-plane buckling. Walls with large story heights between floors that brace the wall in the out-of-plane direction are more vulnerable to buckling. T- or L-shaped wall sections with thin stems may also be more vulnerable. Walls with flanges or other enlarged boundary elements are less susceptible.

5.2.5 Behavior Modes with Little or No Ductility Capacity

The following five behavior modes can be considered to have little or no inelastic deformation capacity:

- Preemptive diagonal tension
- Preemptive diagonal compression (web crushing)
- Preemptive sliding shear
- Preemptive boundary-zone compression
- Preemptive lap-splice slip

The term preemptive is used to indicate that a brittle failure mode preempts any flexural yielding of the wall component. These are force-controlled rather than displacement-controlled behavior modes, as defined in FEMA 273.

a. Strength and Displacement Capacity

During preemptive boundary-zone compression, peak strength is equal to the component flexural strength. For the other four behavior modes of this category, strength will be less than the component flexural strength.

Displacement capacity of these force-controlled behavior modes is limited to the elastic displacement corresponding to peak strength. These behavior modes cannot be considered to have any dependable inelastic displacement capacity.

b. Preemptive Diagonal Tension

Preemptive shear failure of wall components in diagonal tension has been commonly observed after earthquakes. This type of failure typically occurs in components with high flexural strength and inadequate shear reinforcement. The failure is characterized by one or more wide diagonal cracks, which can occur suddenly, with little or no early indication of incipient failure.

c. Preemptive Diagonal Compression (Web Crushing)

Preemptive web crushing is a compression failure caused by high shear forces in the web of a wall section. This behavior mode has been observed in laboratory tests of low-rise flanged walls. Walls with flanges or heavy boundary elements are more prone to this type of failure because larger shear stresses are typically generated in the webs of such sections, as compared to rectangular sections. The web crushing begins at small displacement values, preempting any flexural yielding of the wall.

This failure mode has not been reported in actual structures. Typical buildings do not have foundations with enough overturning capacity to sustain the high forces associated with preemptive diagonal compression failures.

d. Preemptive Sliding Shear

Preemptive sliding shear is most likely to occur in low-rise wall piers that have poor construction joints. Before flexural strength can be reached in such walls, sliding occurs along the surfaces of the construction joint.

e. Preemptive Boundary Zone Compression

This behavior mode only occurs in walls with unusually high axial load — above the balance point considering a maximum concrete strain of 0.004 or 0.005. Such conditions typically occur only in T- or L-shaped sections where the stem of the section is in compression and has inadequate boundary confinement.

f. Preemptive Lap-Splice Slip

Wall behavior governed by preemptive lap-splice slip has not been widely reported. Lap-splice lengths may need to be unusually short for splice failures to occur without prior flexural cyclic behavior. Damage in this behavior mode would be characterized by splitting cracks at lap splices and eventual rocking of the wall component on a crack across the lap-spliced section.

5.2.6 Foundation Rocking Response

Foundation rocking of walls and wall piers is usually a ductile mode of behavior. Capacities depend on the foundation type and geometry and the properties of the soil material. Displacement capacity is normally very high, but the effects of large foundation movements on the superstructure must be considered. FEMA 273 and ATC-40 provide detailed recommendations for foundation components.

5.3 Reinforced Concrete Evaluation Procedures

5.3.1 Cracking

The Component Damage Classification Guides require the user to distinguish between flexural cracks and shear cracks, to identify vertical cracking in the compression zone of wall piers, and to identify horizontal cracking in the compression zone of wall spandrels. The guides also require the user to identify cracks that may indicate lap-splice slipping.

The guides require the user to determine crack widths, which is a factor in assessing the severity of earthquake damage in reinforced concrete wall components.

a. Flexural and Shear Cracks

Flexural cracks are those that develop perpendicular to flexural tension stresses. In wall piers, flexural cracks run horizontally; in wall spandrels, the cracks run vertically. Flexural cracks typically initiate at the extreme fiber of a section and propagate towards the section's neutral axis. For components that have undergone cyclic earthquake displacements in both directions, opposing flexural cracks often join with each other to form a relatively straight crack through the entire section.

Shear cracks are those that result from diagonal tension stresses corresponding to applied shear forces. The cracks run diagonally, typically at an angle of 35° to 70° from the horizontal. The angle of cracking depends on normal forces (e.g., axial load) and on the geometry of the component. For components that have undergone cyclic earthquake displacements of similar magnitude in both directions, the cracks cross each other, forming X patterns.

Flexural cracks often join up with diagonal shear cracks. A typical case is in a wall pier where a horizontal crack at the wall boundary curves downward to become a diagonal shear crack as it approaches the pier centerline. When shear cracks connect to flexural cracks, determine the widths of the flexural portion of the crack and the shear portion of the crack separately.

Cracks initially form perpendicular to the direction of the principal tension stresses in a section. At any point of a component, it is possible to relate the orientation of initial cracking to the applied stresses by considering the stress relationships represented by Mohr's Circle. However, after initial cracking, the orientation of principal stresses will change and crack patterns and stress orientations are affected by the reinforcement.

b. Full-Thickness versus Partial-Thickness Cracking

In investigating reinforced concrete wall components, the engineer should establish whether critical flexural and shear cracks extend through the thickness of the wall. The Component Damage Classification Guides are written under the assumption that the most significant flexural and shear cracks are full-thickness cracks having a similar crack width on each side of the wall.

Laboratory tests on walls have invariably used in-plane loading. Therefore, significant cracks observed in these studies are typically full-thickness. In actual buildings, out-of plane forces and wall deformations may cause cracks to be partial-thickness, or they may result in cracks that remain open to a measurable width on one wall face, but are completely closed on the opposite wall face. In such cases, the engineer should use judgment in assessing the consequences of the critical cracks. It may be justified to use the average of the measured crack width on each face of the wall. More conservatively, the maximum crack width on either face of the wall can be used in the Component Damage Classification Guides.

c. Cracking as a Precursor to Spalling

In the compression region of wall components, cracks occur as a precursor to concrete spalling. Such cracks form parallel to the principal compression stresses, and they may develop when compressive strains in the concrete exceed 0.003 to 0.005. Such cracking typically signals an increased damage severity in the Component Damage Classification Guides. This type of cracking occurs (1) at the boundary regions of component plastic-hinge zones for flexural behavior, and (2) under a diagonal-compression (web-crushing) type of shear failure.

For wall piers in flexure, this type of cracking is vertical. For wall spandrels in flexure, the cracks are horizontal. In both cases, the cracks occur near the extreme fibers of the section in the plastic hinge zone(s). Such cracking is less likely in spandrels because of the absence of axial load.

The cracking in compression regions of flexural members could appear similar to splitting cracks resulting from lap-splice or bond slip of the reinforcement. Both types of cracking tend to occur in the boundary regions of plastic-hinge zones. Some distinguishing features of the two different types of cracks are described below:

Cracks as a precursor to spalling in the compression region:

- Occur under conditions of high compressive strain.
- Cracks may be relatively short. Sounding with a hammer (See Section 3.8) may reveal incipient spalling.
- Cracks occur at the extreme fibers of the section, typically within the cover of the concrete.

Bond or lap-splice splitting cracks:

- Occur at the locations of longitudinal reinforcement that is susceptible to bond or lap-splice slip. (Large bar diameters or inadequate lap-splice length.)
- Cracks tend to be relatively long and straight, mirroring rebar locations. The cracks originate at the reinforcement and propagate to the concrete surface.

Diagonal cracking in the web of the wall can be a precursor to a diagonal-compression (web-crushing) shear behavior. Unlike diagonal tension cracks, these cracks may not open widely, but under increasing damage, the cracks will be followed by spalling of the web concrete. This occurs because the compressive strength of concrete reduces in the presence of transverse tensile strains.

d. Splitting Cracks at Lap Splices

If lap splices are insufficient to develop the required tension forces in the reinforcement, slip occurs at the splices. The visible evidence of lap-splice slip is typically longitudinal cracks (parallel to the splice) that originate at the lap splice and propagate to the concrete surface. Thus, the crack locations reflect the locations of the lap-spliced reinforcement.

e. Crack Widths

Crack widths are to be measured according to the investigation procedures outlined in this document. In the Component Damage Classification Guides, the maximum crack width defines the damage severity. When multiple cracks are present, the widest crack of the type being considered (e.g., shear or flexure) governs the damage severity classification.

The maximum crack width may be significantly larger than the average width of a series of parallel cracks. Although average crack width may be a better indicator of average strain in the reinforcement, maximum crack width is judged to be more indicative of maximum reinforcement strain, and, in general, damage severity. A concentration of strain at one or two wide cracks typically indicates an undesirable behavior mode and more serious damage, whereas an even distribution of strain and crack width among numerous parallel cracks indicates better seismic performance.

The crack width criteria in the Component Damage Classification Guides are based on a comparison to research results, rather than on detailed analyses of crack width versus strain relationships. The criteria recognize that the residual crack width observed after an earthquake may be less than the maximum crack widths occuring during the earthquake.

5.3.2 Expected Strength and Material Properties

a. Expected Strength

The capacity of reinforced concrete components is calculated initially using expected strength values. Expected strength is defined in Section 6.4.2.2 of FEMA 273 and Section 9.5.4.1 of ATC-40 as "the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected."

Expected component strength may be calculated according to the procedures of ACI 318 — or other procedures specified in this document — with a strength-reduction factor, ϕ, taken equal to 1.0. Expected material strength rather than specified minimum material strength is used in the calculations. Material strength values are discussed below.

b. Reinforcing Steel Strength

Tables 6-1 and 6-2 of FEMA 273 gives the specified yield and tensile strength of reinforcing steel that has been used in buildings since the turn of the century. ASTM A432 reinforcing steel, with a specified yield strength of 60 ksi, was introduced in 1959 (CRSI Data Report 11). Prior to this date, reinforcing steel typically had a specified yield strength of 40 ksi or less.

The actual yield strength of reinforcing steel typically exceeds the specified value, as discussed in Section 9.5.4.1 of ATC-40. Tests by Wiss, Janney, Elstner Associates (1970) on ASTM A432 Grade-60 bars showed an average tensile stress of 67 ksi for bars at a strain of 0.005 and an average tensile stress of 70 ksi for bars at a strain of 0.008. Stresses were based on actual rather than nominal bar areas, and the standard deviation was about 7 ksi. Similarly, data cited by Park (1996) indicate that actual bar yield strength averages about 1.15 times the specified value. At moderate-to-high ductilities, strain hardening will further increase the stress in yielding reinforcement.

FEMA 273 (Section 6.4.2.2) and ATC-40 prescribe that expected strength values be calculated assuming a strength of yielding reinforcement equal to "at least 1.25 times the nominal yield strength." For this document, in the absence of applicable test data, the initial expected strength of yielding reinforcement, f_{ye}, is assumed equal to 1.25 times the nominal yield strength. A range of reinforcement strength, between 1.1 and 1.4 times the nominal yield strength, can also be considered in the evaluation procedures, should field observation warrant.

Section 6.3.2 of FEMA 273 gives recommendations for establishing reinforcing steel strength by testing.

c. Concrete Strength

Table 6-3 of FEMA 273 gives typical concrete compressive strength values that may be assumed in buildings according to year of construction and structural member type. These values can be considered as specified or nominal values, f_c', rather than expected values, f_{ce}'. If structural drawings are available that indicate specified concrete strengths, use the values from the drawings instead of the assumed values from the table.

The actual concrete strength in existing structures can significantly exceed the specified minimum concrete strength, by factors of up to 2.3 (Park, 1996). For this document, in the absence of applicable test data, the initial expected concrete compressive strength is assumed to be equal to 1.5 times the specified strength. A range of concrete strength, between 1.0 and 2.0 times the specified strength, can also be considered in the evaluation procedures, when based on field observations. FEMA 273 and ATC-40 do not specifically address the relationship between expected and specified concrete strengths.

In the case that concrete compressive strength test results from the existing construction are available, use these results to establish the expected concrete strength. The expected concrete strength considers the likely strength increase of the concrete over time, as is discussed in Section 9.5.2.2 of ATC-40. In the absence of more specific data, the initial expected strength can be taken equal to 1.2 times the tested strength at 28 days after construction.

Section 6.3.2 of FEMA 273 gives recommendations for establishing concrete compressive strength by testing.

Concrete strength seldom has a significant effect on wall flexural strength. It will have a more significant effect on shear strength, one component of which is taken proportional to $\sqrt{f_{ce}'}$.

d. Concrete Modulus of Elasticity

The modulus of elasticity for concrete is calculated according to ACI-318, using the expected concrete strength as defined above.

5.3.3 Plastic-Hinge Location and Length

Plastic hinges occur at the critical flexural regions of wall members where moment demand reaches moment strength. For earthquake-induced forces, plastic hinges typically occur at the face of a supporting member or foundation. Lap splices may force plastic hinges to develop or concentrate at the ends of the lap-splice length.

Potential plastic-hinge locations are to be identified for all wall elements subjected to earthquake forces and displacements. The locations are established as part of identifying the governing mechanism for nonlinear lateral deformation of the structure, as described in Section 2.4.

Isolated walls or stronger wall piers (component type RC1) typically have a single plastic hinge region at the base of the wall. A plastic hinge could also occur above the base of a wall at a location of reduced strength such as (1) a setback of the wall, (2) a level where a substantial amount of the vertical reinforcement is curtailed, or (3) a level above which the number of walls resisting seismic forces is reduced.

The curtailment of reinforcement may need to be investigated in some detail. Plastic hinging may occur at an area of reinforcement curtailment because (1) higher mode effects cause a moment at that level which exceeds the moment diagram assumed in design, and (2) designers may not have extended reinforcement "beyond the point at which it is no longer required to resist flexure for a distance equal to the flexural depth of the member" as is required by ACI 318 (1995).

Weaker wall piers (component type RC2) under flexural behavior develop plastic hinges at the top and bottom regions of the component, typically at the face of the connecting spandrel or foundation component. Similarly, weak spandrels (component type RC3) under flexural behavior develop plastic hinges at each end of the component, typically at the face of the connecting wall piers.

Plastic hinges are developed only for the ductile flexural behavior mode or for behavior modes with intermediate ductility capacity in which flexure initially governs response. Wall components governed by preemptive shear failures or foundation rocking do not exhibit plastic hinging (although, for foundation rocking, the soil beneath the foundation could be treated conceptually as the plastic-hinge region).

Plastic-hinge lengths define the equivalent zones over which nonlinear flexural strain can occur. The length of plastic hinging generally depends on the depth of the member and on the moment-to-shear ratio (M/V). Bond conditions of the reinforcement also affect the length over which yielding occurs and the penetration of reinforcement yielding into the supporting member.

For reinforced concrete, equivalent plastic-hinge length can be roughly estimated as equal to one-half the member depth (Park and Paulay, 1975). A similar estimate is applied to walls in Section 6.8.2.2 of FEMA 273, where l_p is "set equal to one half the flexural depth, but less than one story height." The 1997 UBC (ICBO, 1997) states that l_p "shall be established on the basis of substantiated test data or may be alternatively taken as $0.5l_w$."

Based on research specifically applicable to walls, the equivalent plastic-hinge length, l_p, can be set at 0.2 times the wall length, l_w, plus 0.07 times the moment-to-shear ratio, *M/V* (Paulay and Priestley, 1993).

Equivalent plastic-hinge length, as calculated above, is used to relate plastic curvature to plastic rotation and displacement. The actual zone of nonlinear behavior may extend beyond the equivalent plastic-hinge zone.

The Component Damage Classification Guides refer to the plastic hinge length to identify the zone over which nonlinear flexural behavior and damage may be observed. The expected zone of inelastic flexural behavior and damage in the Component Damage Classification Guides can be taken as two times l_p.

In short spandrel beams, the plastic zones at the ends of the beam may merge. In diagonally reinforced coupling beams, the entire length of the spandrel will yield.

5.3.4 Ductility Classifications

In these evaluation procedures, ductility capacity and demand are classified as either low, moderate, or high. The following approximate relationship can be assumed:

Classification	Displacement Ductility
Low Ductility	$\mu_\Delta < 2$
Moderate Ductility	$2 \le \mu_\Delta \le 5$
High Ductility	$\mu_\Delta > 5$